ARIZONA DEPARTMENT OF TRANSPORTATION

ADOT

REPORT NUMBER: FHWA-AZ89-260

SIZING RIPRAP FOR THE PROTECTION OF APPROACH EMBANKMENTS & SPUR DIKES AND LIMITING THE DEPTH OF SCOUR AT BRIDGE PIERS & ABUTMENTS

Volume I: Literature Review & Arizona Case Histories

Prepared by:
Simons, Li & Associates, Inc.
4600 South Mill Avenue, Suite 280
Tempe, Arizona 85282

June 1989

Prepared for:
Arizona Department of Transportation
206 South 17th Avenue
Phoenix, Arizona 85007
in cooperation with
U.S. Department of Transportation
Federal Highway Administration

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catatlog No.
FHWA-AZ89-260, I		

4. Title and Subtitle	5. Report Date
SIZING RIPRAP FOR THE PROTECTION OF APPROACH EMBANKMENTS AND SPUR DIKES AND LIMITING THE DEPTH OF SCOUR AT BRIDGE PIERS AND ABUTMENTS	June, 1989
	6. Performing Organization Code
7. Author(s)	**8. Performing Organization Report No.**
Dr. Ruh-Ming Li, Dr. Robert MacArthur, George Cotton	
9. Performing Organization Name and Address	**10. Work Unit No.**
Simons, Li & Associates, Inc. 4600 South Mill Avenue, Suite 280 Tempe, Arizona 85282	11. Contact or Grant No. HPR-PL-1(31) Item 260
12. Sponsoring Agency Name and Address	**13. Type of Report & Period Covered**
ARIZONA DEPARTMENT OF TRANSPORTATION 206 S. 17TH AVENUE PHOENIX, ARIZONA 85007	Final, Feb. 1987 - June, 1989
	14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the U.S. Department of Transportation, Federal Highway Administration

16. Abstract

This report presents a review of published literature on riprap design technology and examines Arizona case histories of riprap performance. The literature review grouped the factors affecting riprap design into hierarchical categories relative to scale. The four factors identified include: riprap properties, site characteristics, hydraulic and sediment transport conditions, and river response. Eleven case histories from documentation supplied by the Arizona Department of Transportation (ADOT) and the U.S. Department of Agriculture, Soil Conservation Service (SCS) are examined. The review of Arizona case histories is intended to provide the basis for understanding the dominant river processes associated with riprap protection measures. The literature review and case histories indicate a set of design requirements to be considered when designing riprap revetment.

Design Procedure, Volume 2

23. Registrant's Seal

Registered Professional Engineer (Civil)
CERTIFICATE NO. 21560
DENNIS L. RICHARDS
Date Signed 7/7/89
ARIZONA, U.S.A.

17. Key Words	18. Distribution Statement
Riprap, scour, shear stress tractive force, safety factor, bank protection	Document is available to the U.S. public through the National Technical Information Service, Springfield, Virginia 22161

19. Security Classification (of this report)	20. Security Classification (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	131	

TABLE OF CONTENTS

VOLUME I

LIST OF FIGURES

LIST OF TABLES

LIST OF SYMBOLS

Symbol	Description
C	Froude Number coefficient for relative depth
C_s	Froude Number coefficient relating Shields parameter to the relative depth
d	average depth of flow
D_{15}	rock size which only 15% is finer by weight
D_{20}	rock size which only 20% is finer by weight
D_{30}	rock size which 30% is finer by weight
D_{35}	rock size which only 35% is finer by weight
D_{50}	median rock size
D_{100}	rock size which 100% is finer by weight
D_m	representative riprap grain size
D_{max}	maximum riprap size specified in the design gradation (D_{100})
F	Froude Number, V_a/ gd
g	gravitational constant, 32.2 ft/sec^2
G	gradation coefficient, $1/2[D_{84}/D_{50} + D_{50}/D_{16}]$
K	tractive force ratio
r	the radius of curvature
R	hydraulic radius
S	slope of the energy grade line
V_a	average velocity
w	topwidth of the channel
W_{100}	rock weight of gradation which 100% are lighter
W_{15}	rock weight of gradation which 15% are lighter
W_{50}	rock weight of gradation which 50% are lighter
γ	unit weight of water, 62.4 lbs/ft^3
τ	average boundary shear stress
τcb	critical shear stress on the bed of the channel
τcs	critical shear stress on the side-slope of the channel
θ	angle of repose of the material that forms the side-slope
ϕ	channel side slope angle

I. INTRODUCTION

This report presents the results and findings of Task One of Research Project No. HPR-PL-1(31) Item 260, Sizing Riprap for the Protection of Approach Embankments and Spur Dikes and Limiting the Depth of Scour at Bridge Piers and Abutments. The objective of this study task was to perform a literature search, to identify the research that has been conducted on riprap protection, with an emphasis on research pertaining to conditions in Arizona. In formulating the approach for the study, it was determined that the initial review phase should address not only published research, but should also seek out case histories of riprap performance. Examination of Arizona case histories is intended to provide the basis for understanding the dominant river processes associated with riprap protection measures. It was felt that combining published research on riprap performance with information from case histories would best allow the determination of riprap design requirements for conditions characteristic of Arizona.

Case histories were sought from a number of Federal, State, County and local agencies during Task One. The agencies contacted expressed a willingness to share design experience and practice. All districts of the Arizona Department of Transportation (ADOT) were contacted for information on their knowledge of riprap problems. An extensive review of reports, construction plans, and bridge inspection records was conducted at ADOT headquarters with the assistance of the hydraulics and structures sections staff. We found ADOT's evaluation of deficiencies at bridge structures related to scour to be a very pertinent source of case histories. Over the past six years, the Scour Team has evaluated scour conditions at over one hundred bridge sites, and has prepared a substantial number of reports, and initiated projects to construct countermeasures.

Contact with Federal agencies included: The Corps of Engineers, Bureau of Reclamation and Soil conservation Service. Discussions with the staff at these agencies lead us to the conclusion that the Soil Conservation Service could supply the most pertinent case histories. Background on the type of information available from each of these federal agencies contacted is discussed later in the report. The Central Arizona Water Conservancy District was contacted and the Salt River Project. The design problems encountered by these agencies were sufficiently different from the focus of this study that they were not pursued. The Pima County Department of Transportation and Flood Control District and the Flood Control District of Maricopa County were contacted. Neither of these agencies uses riprap to any great extent; soil cement and gabions are preferred for most projects. The Cities of Phoenix and Tucson were contacted and as with their counterparts at the county level, soil cement is the preferred method of stabilizing river banks.

Eleven case histories were developed from documentation supplied by ADOT and the SCS. Eight of the ten case histories are from ADOT projects and cover countermeasures installed at bridge waterways. Two SCS projects are presented as case histories.

The literature search concentrated on four catagories of channel stability: riprap characteristics, hydraulic and sediment transport conditions, site characteristics, and river response. The review provides an overview of research pertinent to the study.

The literature review and case histories point to a set of design requirements that should be considered for riprap protection. The second volume of this report addresses methodologies currently available to meet these design requirements. The limitations of these methods and particularly their applicability to conditions observed in Arizona were evaluated and an interim design procedure is recommended.

II. REVIEW OF LITERATURE ON RIPRAP DESIGN TECHNOLOGY

2.1 Overview of Literature on Riprap Design Technology

The design of riprap protection measures involves assessment of a number of factors associated with the river environment, the bridge site, and the quality of the riprap material. As can be seen from the case histories presented in Chapter 4, most bridge sites are affected by a combination of these factors. There is a body of research that addresses individual aspects of riprap design, where data on riprap performance has been gathered from laboratory studies. Another body of research has addressed field performance of riprap installations. Field study requires a longer period of investigation, and physical measurements are more difficult to accurately obtain, and therefore, are less commonly reported in the literature.

The literature reviewed for this study has been grouped into the following four catagories:

Riprap Properties:

Size, gradation, shape, layer thickness, density, rock durability, and bedding requirements.

Site Characteristics:

Structure location (encroachment length and skew), channel alignment and shape, and bank side-slopes.

Hydraulic and Sediment Transport Conditions:

Incipient motion, boundary shear stress, local scour, general aggradation/degradation, bed forms.

River Response:

Change in channel area, topwidth, depth, gradient, bed-material gradation, and sinuosity in response to flood flows.

This grouping of factors in riprap design is hierarchical in scale, that is one set of factors addresses processes that are on the order of a few feet, while others may be on the order of tens of miles. Riprap characteristics involve the population of riprap particles, which are each less than a few feet in size. Site characteristics are concerned with a scale on the order of two to three times the crossing length, or typically on the order of a few hundred feet. Hydraulic and sediment transport conditions are typically evaluated over a reach length, upstream and downstream of the site, of a few thousand feet. River response is typically evaluated at the basin level on the scale of several tens of miles. This distinction in scale is not always easily perceived, but both large scale and small scale factors can lead to design deficiencies for a project.

2.2 Riprap Characteristics

The physical characteristics of the rock particles that make up riprap protection most often sited in specifications include: a characteristic size, gradation, layer thickness, shape, specific gravity, durability, and filter requirements. Research on these basic physical characteristics has concentrated primarily on size, gradation, shape, and layer thickness.

Characteristic Size

The characteristic riprap size is generally taken as the diameter of the median of the gradation by weight or the D_{50}. General references on riprap design, such as Sediment Transport Technology (Simons and Senturk (1977)), present a number of design procedures, the majority of which characterize the riprap by the D_{50} size. In the training and design manual, Highways in the River Environment, (Richardson, et al., 1987), it is noted that riprap may armor, "...leaving a layer of large rock sizes which cannot be transported under the given flow conditions. Thus, the size of rock representative of the stability of the riprap is determined by the larger sizes of rock. The representative grain size D_m for riprap is larger that the median rock size D_{50}." Using the recommended gradation in Highways in the River Environment, (page V-26,27) where the $D_{20} = 1/2\ D_{50}$ and $D_{100} = 2\ D_{50}$, an effective grain size of 1.25 D_{50} is computed which corresponds to the D_{65} riprap size. The manual goes on to note that, "[T]he weight of a bed-material particle is important to the stability of the particle. Thus, it is more meaningful to compute the representative particle size based on weight of the particle than on its diameter." Mahmood (1973), found that the distribution of bed-material properties could be described by a log-normal probability distribution. The representative size of the bed material based on the weight of the particles can be described as a function of the gradation coefficient (Mahmood, 1973):

$$D_m = D_{50} \exp \left[\frac{3}{2} (\ln G)^2\right]$$

where

D_m = the representative grain size,

D_{50} = median rock size, and

$G = 1/2\ [D_{84}/D_{50} + D_{50}/D_{16}]$.

which is always greater than one for a non-uniform grain size distribution.

More recently, data gathered by Maynord (1986) indicates that the D_{50} may not characteristic size riprap stability. He found that for the range of gradations tested by the Corps of Engineers Waterways Experiment Station, that incipient failure of riprap could be more reliably evaluated using the D_{30} size. In support of this finding, Maynord sites work on bed armoring by Shen and Lu (1983) and the Einstein bed-load function (1950) which uses D_{30} and D_{35} as characteristic sizes, respectively.

Gradation

The gradation of riprap sizes is of considerable importance both in terms of the stability and in preventing leaching of the base material. Anderson (1970) noted that with a graded distribution of riprap as the thickness is increased, the interstices left by large particles are filled by smaller particles. As the layer thickness or the variations in particle sizes increases, the number of direct paths to the base material decreases. When boundary shear stress at the riprap surface is less than the smaller sizes in the distributions, the stability of the riprap is maintained. A riprap gradation with a large variation in particle sizes was observed by Anderson to experience erosion of the smaller sizes, as boundary shear increased. In riprap gradations with less variation in particle size, the smaller particles tended to be sheltered by larger particles and remained stable as boundary shear stress increased. Gradations tested by Anderson ranged from uniform to $G = 2.0$. Highways in the River Environment recommends using the following U.S. Army Corps of Engineers (1982) criteria for establishing gradation limits for riprap:

- The lower limit of D_{50} stone should not be less than the size of stone required to withstand the design shear forces.
- The upper limit of D_{50} stone should not exceed five times the lower limit of D_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.
- The lower limit of D_{100} stone should not be less than two times the lower limit of D_{50} stone.
- The upper limit of D_{100} stone should not exceed five times the lower limit of D_{50} stone, the size which can be obtained economically from the quarry, or the size that satisfies layer thickness requirements.
- The lower limit D_{15} stone should not be less than one sixteenth the upper limit of D_{100} stone.
- The upper limit of D_{15} stone should not be less than the upper limit of the filter material.
- The bulk volume of stone lighter than the D_{15} stone should not exceed the volume of voids in the structure without this lighter stone.

Murphy and Grave (1963) tested various rock sizes and gradations in conjunction with protection of overflow dikes. Two gradations, A and Al (Figure 1), failed under the same conditions although gradation A had maximum particles 36 inches in diameter as opposed to 24 inches for gradation Al. Both gradations had a median diameter of 16 inches. The two gradations B and C, failed under the same conditions. The greater variation in particle sizes in the C gradation resulted in a maximum size of 24 inches compared to 16 inches for the B gradation. However, only 15 percent of the B gradation was less than half the D_{50}, compared to 30 percent for the C gradation. In the model test, it was found that riprap failure occurred by removal of smaller particles, resulting in the dislodgement of larger particles. Murphy and Grace concluded that stones larger than some critical size (approximately D_{65} in their tests), do not increase riprap stability.

Searcy (1967) proposed three classes of riprap for use in riprap protection at highway bridges and proposed a single gradation. The gradation is referenced to the median size, D_{50}.

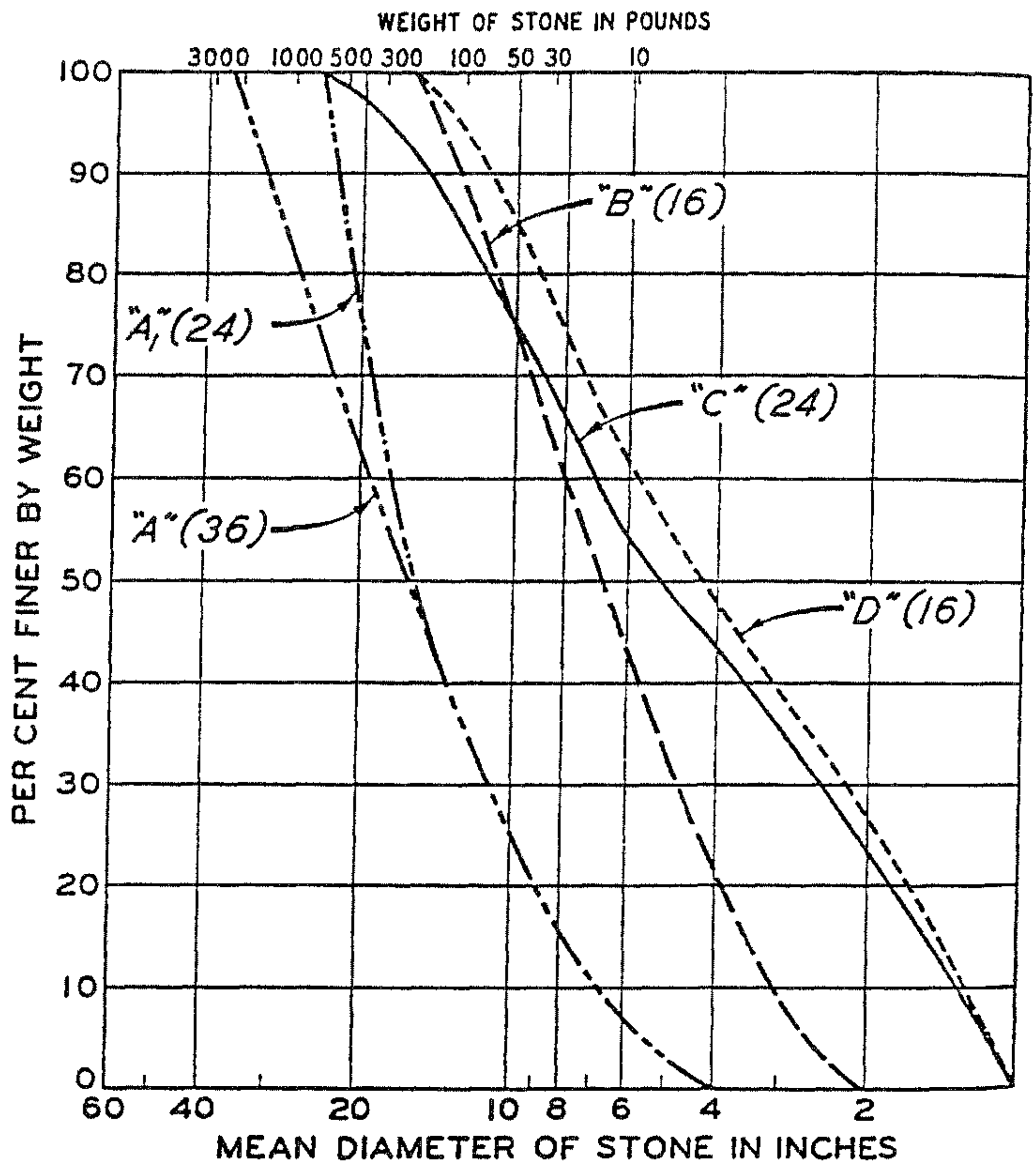


Figure 1. Riprap Gradations Tested by Murphy and Grace (1963)

Size of Stone	Percent of total weight smaller than the given size
$3D_{50}$	100
$2D_{50}$	80
$1D_{50}$	50
$0.1D_{50}$	10

Searcy based this gradation on findings by Murphy and Grace, but realized that unless a large quantity of riprap was to be installed, that it might prove undesirable to specify more than a single gradation. The Searcy gradation was intended to accommodate actual field conditions.

In the Corps of Engineers design manual, "Hydraulic Design of Flood Control Channels" EM-1601, (1970), a set of criteria was presented for establishing gradation limits. The criteria result in a range of stone weights for each fraction of the gradation rather than a single gradation curve. Ranges are determined for the D_{100}, D_{50}, and D_{15} size fractions, where the lower limit for D_{50} is set to meet boundary shear stress conditions, and the upper limit is set based on an economically feasible quarry size. The limits for the other two size fractions are set as follows:

$$W_{100L} > 2\ W_{50L}$$

$$W_{100U} < 5\ W_{50L}$$

$$W_{15L} > 1/16\ W_{100U}$$

$$W_{15U} < W_{50U}$$

where W is the stone weight and the numerical subscript refers to the percent lighter by weight, and "L" and "U" denoting the upper and lower limit of the range.

Maynord (1986) reports on laboratory tests conducted by the Crops on riprap and indicates that for gradations having D_{85}/D_{15} less than 4.6, a single incipient failure criteria could be developed. As mentioned earlier, Maynord found the D_{30} size to be characteristic of riprap stability.

To make the specification of riprap gradation somewhat easier, the Crops issued Engineer Technical Letter (ETL) No. 1110-2-120 that provides additional guidance for riprap channel protection. This ETL provides a series of tables that allow gradation limits to be determined based on physical characteristics of the riprap.

Blodgett and McConaughy (1986) compare stone gradations specified in different design procedures. Figure 2 presents their comparison and includes Oregon and California specifications.

Shape

Another important property for riprap stability is riprap particle shape. Angular, well-proportioned rock particles tend to interlock and form a more stable mass than rounded rock shapes. Lane (1955) observed the angle of repose of material on stock piles and noted that the angle of repose increased for angular and crushed rock over round rock. Lane constructed a chart showing the angle of repose as a function of shape and median riprap diameter. Simons (1957) developed a similar set of curves based on his observations of the angle of repose for coarse, noncohesive material. The importance of the angle of repose in the stability of riprap was shown theoretically by Carter, Carlson and Lane (1953) which they expressed as the tractive force ratio, K,

$$K = \frac{\tau cs}{\tau cb} = 1 - \frac{\sin^2 \phi}{\sin^2 \Theta}$$

where

τcs = critical shear stress on the side-slope,

τcb = critical shear stress on the bed,

ϕ = channel side-slope angle,

Θ = the angle of repose of the material that forms the side-slope.

Stevens and Simons (1971), associated the angle of repose to the moment resisting overturning of a riprap particle, as part of their development of a safety factor for riprap design.

Most specifications for riprap shape recommend angular stones, and in addition give ratios for length and breadth of the stone relative to its length. The basic rule for riprap proportion, (Searcy, 1967), which is widely used is that "neither breadth nor thickness of a single stone should be less than one-third its length." In EM-1601, the Crops also requires "not more than 25 percent of the stones, reasonably well distributed throughout

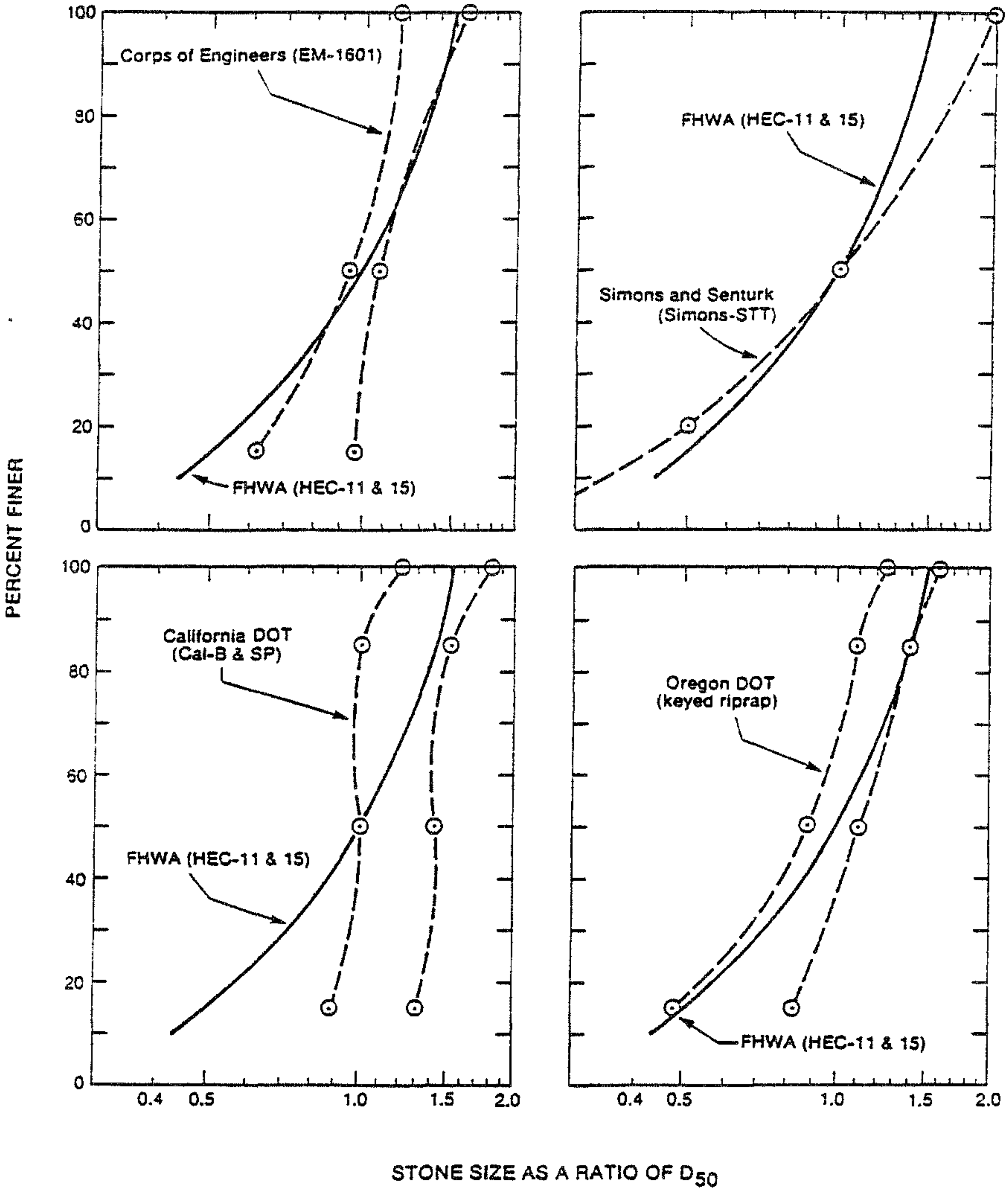


Figure 2. Comparison of Specified Riprap Gradations, Blodgett and McConaughy (1986)

the gradation, shall have a length more than 2.5 times the breadth and thickness."

Thickness

The general rule for riprap thickness is that all stone sizes should be contained within the layer thickness. This results in a thickness equal to the diameter of the largest riprap particles in the distribution. Simons and Senturk (1977), the Corps of Engineers (EM-1601, 1970), Searcy (1967), and others use this rule. Stevens, Simons and Richardson (1984), recommend that in the case of riprap with a large gradation coefficient ($G > 3.0$) that the thickness should be increased to 1.5 D_{100} to provide enough material for armor-plating. Maynord (1986) showed increased riprap stability as thickness increased up to 1.5 D_{100}. The Corps data as presented by Maynord shows that increased riprap thickness decreases the required size.

Highways in the River Environment recommends the riprap thickness should not be less than twelve inches for practical placement, less than the diameter of the upper limit of the D_{100} stone, or less than 1.5 times the diameter of the upper limit D_{50} stone, whichever is greater. If riprap is placed underwater, the thickness should be increased by 50 percent; and if subject to attack by large floating debris or wave action, it should be increased six to twelve inches.

Density

The rock density used to form riprap is a basic factor in riprap stability. However, the variation in density among natural rock types suitable for use as riprap is small. The specific gravity of riprap composed of quartz and feldspathic minerals is 2.65. A minimum specific gravity of 2.5 is often specified.

Durability

The durability of riprap is important both during the transportation of riprap particles from quarry to construction site and during in-service performance. Evaluation of rock durability depends on geotechnical techniques and geologic concepts which include site evaluation, field testing, and laboratory tests. Common laboratory tests include: Los Angeles

Abrasion, Point-load test, Schmidt hammer, freeze-thaw test, sulfate soundness test, and slake durability-two cycle (ASTM, 1980). Summer and Johnson (1982) devised a rock durability flow chart, which provides a procedure for evaluating rock suitability as riprap for channel lining. This procedure incorporates both site investigation and laboratory testing as required, and is a simple step-by-step approach (Figure 3). Smith, McCauley and Mearns (1970) studied quality control of riprap by the California Division of Highways and recommended the durability absorption ratio (DAR) as the best means of combining the results of inexpensive laboratory tests into an index usable for specifying riprap durability.

Bedding Requirements

The importance of using a filter medium to separate the channel bank material from the overlying riprap gradation has been stressed since the 1940's. Use of a graded rock filter blanket was proposed by Terzaghi (1948) and thoroughly tested by the Corps of Engineers Waterways Experiment Station (1941, 1948). The Terzaghi filter gradation is routinely specified and advocated by some (Posey, 1957) to be the only acceptable filter for permanent riprap installations. However, the cost of producing the Terzaghi filter gradation and the difficulty of installing rock filter blanket has lead to a preference for other filter materials, particularly the synthetic fabrics (Dellaire, 1977).

Since the introduction of synthetic fabrics in the late 1950's, there has been substantial interest in the many possible geotechnical applications of this technology, among which is as a filter medium. Development of design criteria and specifications for geotechnical fabrics has advanced through research by the Corps of Engineers and Federal Highway Administration. Initial research by the Waterways Experiment Station (1972) pointed out that few engineering properties of plastic filter cloth were known at the time, but that good performance had been documented under severe loading conditions. Concern was expressed over the lack of permeability of the fabrics by WES (White, 1982) in their documentation of the performance of filter fabrics in conjunction with bank protection measures. The Federal Highway Ad-ministration (Bell and Hicks, 1980) compiled literature and field performance data which resulted in the development of interim criteria and

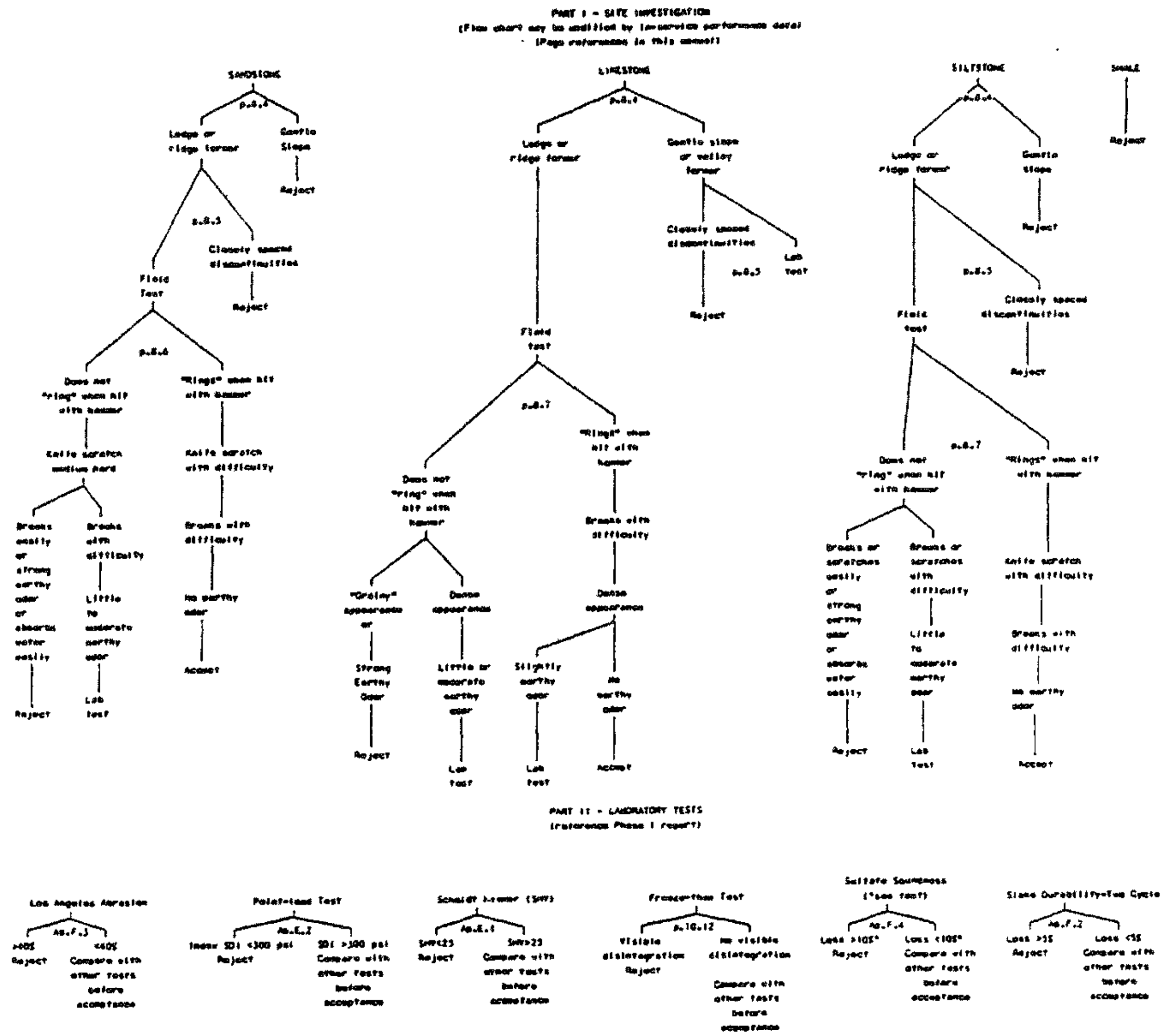


Figure 3. Rock Durability Flow Chart, Summers and Johnson (1982)

specification of fabric properties needed for a wide range of highway applications. Riprap protection is not specifically addressed in this study but related applications such as filtering and separation are pertinent. Christopher (1983) reports on two riprap installations that are over a decade old constructed in 1969 in Florida. This leaves the question of long-term performance of synthetic fabrics still open.

2.3 Hydraulic and Sediment Transport Conditions

The stability of riprap at a site can depend both on the hydraulic forces to which the individual riprap particles are subject and on movement of the channel boundary. The behavior of riprap in a flow field has been studied by a number of researchers. Incipient motion of riprap particles in a uniform flow field has probably been the most widely studied aspect of riprap stability. Nonuniform flow conditions that have been studied include: flow in channel bends and zones of expanding or contracting flow (conditions that are characteristic of flow near bridge abutments, guidebanks, and piers). Movement of the channel boundary can take place due to local scour at piers, abutments and near spurs; or from more general changes due to a change in the sediment transport capacity in the channel reach where the structure is located. The regime of a moveable bed channel and the associated bed-forms can also be an important factor.

Incipient Motion

The flow condition which just sets a solid particle in motion is the primary criteria used in riprap design. Shields (1936) conducted experiments with uniform sediment sizes to develop his well-known incipient motion diagram, which is shown in Figure 4. The Shields diagram is a nondimensional chart with the vertical axis being the ratio of boundary-shear stress to particle weight, and the horizontal axis being the particle Reynolds number. Laboratory data on the incipient motion of nonuniform size distributions has been collected by Gessler (1963) and by Little and Mayer (1972). Gessler (1971) noted that because of fluctuations in turbulence intensity and the nonuniformity of channel bed material, that Shields criteria must be viewed in a probabilistic manner. Shen and Lu (1983) developed a procedure for predicting the final imposition of armoring bed. They found that D_{30} should

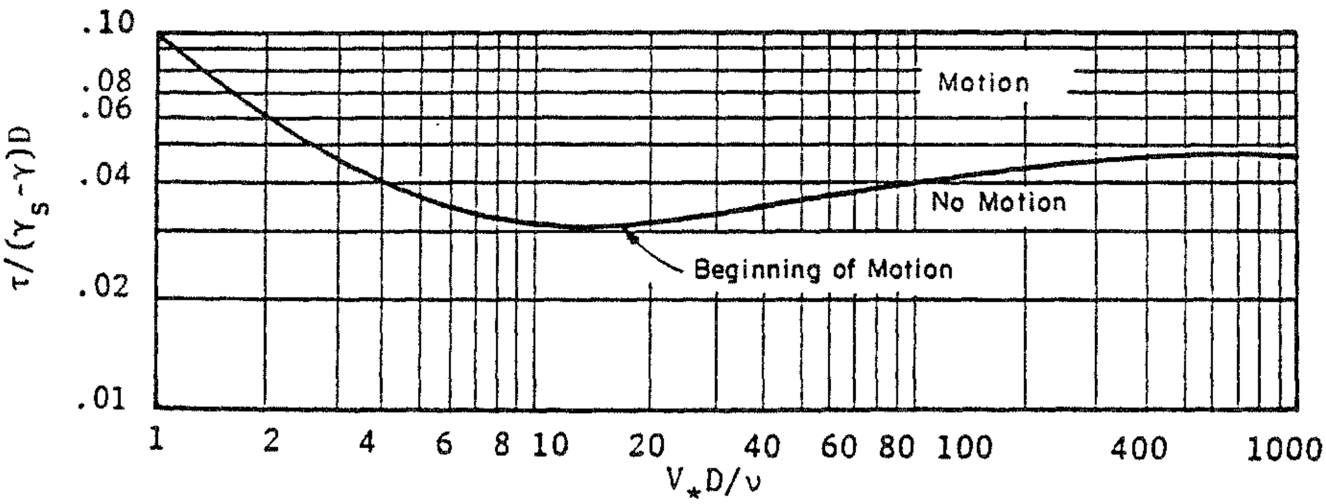


Figure 4. Shields Diagram (after Gessler, 1971)

be used to substitute for the uniform sediment size in the Shields diagram tc describe incipient motion.

Most flow conditions associated with riprap design involve fully turbulent flows with the particle Reynolds number in excess of 100, and therefore, it is commonly assumed for design that the Shields Parameter is a constant value. The following table summarized some of the values of the Shields parameter that have been recommended or incorporated into riprap design procedures.

Source	Shields Parameter
Lane (1955)	0.047
Anderson (1970)	0.039
EM-1601 (1970)	0.040
Gessler (1971)	
95% level	0.024
50% level	0.047
Maynord (1978)	0.037
Maynord (1986)	0.033 to 0.040 (depends on thickness)

By combining the Shields criteria, with the Manning equation, and using the Strickler roughness equation, the following relationship can be derived.

$$\frac{D_{50}}{d} = C\,F^3 \tag{1}$$

where

D_{50} = mean particle size,

C = coefficient as defined below,

d = depth of flow,

F = Froude Number = $\frac{V_a}{gd}$,

V_a = average velocity,

g = gravitational constant.

Maynord (1978) showed that procedures by Anderson (1970), Li et al (1976), Ramette (1963), Em-1601 (1970), and Isbash (1935) can be closely approximated by the above equation. In general, all these procedures have the same exponent as the above equation, but the coefficient varies for each procedure. Maynord gives coefficient values for straight channels ranging from 0.22 for bottom riprap (Factor of Safety of 1.0) to 0.33 for riprap placed on a 2:1 bank (Factor of Safety of 2.0). For curved channel sections, Maynord recommends the following equation for determining the C coefficient:

$$C = 0.70\ (r/w)^{-0.5} \qquad (2)$$

where r = radius of curvature, and
w = topwidth of the channel.

The C coefficient can be converted to a corresponding value of the Shields parameter, C_s, using the following equation:

$$Cs = \frac{1}{(72.3 * C^{2/3})} \qquad (3)$$

Note that large values of C are equivalent to small values of the Shields parameter.

Blodgett and McConaughy (1986) developed a new procedure based on an extensive set of field data. The equation has a dimensional form and is similar to equation (1) but does not explicitly account for flow depth:

$$D_{50} = 0.01 * V_a^{2.44} \qquad (4)$$

The equation represents a lower envelope for field sites that had erosion of riprap particles. The authors also evaluate seven design procedures commonly used in highway engineering. Maynord (1986) re-evaluated hydraulic data on riprap stability and has proposed the following equation:

$$\frac{D_{30}}{d} = 0.53\ C\ F^{2.5} \qquad (5)$$

The C coefficient was found to vary with total riprap thickness. For a riprap thickness equal to the maximum size in the gradation, and riprap place on the bed of a straight channel, C = 0.30. Additional tests are now under way to determine riprap stability on channel side-slopes and in channel bends.

Boundary Shear Stress

In a flow field, a shear stress is developed at the channel boundary as the flow velocity is reduced to zero at the boundary. If the velocity distribution is known for the flow field, then the boundary shear stress can be determined. For turbulent flow, the velocity fluctuates substantially and results in bursts of shear stress higher than average. The boundary shear stress can be determined for relatively simple flow conditions; but for complex flow conditions, it is seldom calculated directly. For uniform flow conditions, the average boundary shear stress is described by the following equation:

$$\tau = \gamma R S \qquad (6)$$

where γ = unit weight of water;
R = hydraulic radius; and,
S = slope of the energy grade line.

Basic research on the distribution of boundary shear stress in straight trapezoidal channels was conducted by Olsen and Florey (1952) and Replogle and Chow (1966). The results of the membrane analysis by Olsen and Florey is widely published in many textbooks and design manuals, and can be used to calculate the distribution of shear stress in a straight trapezoidal channel. In more complicated flow conditions such as bridge crossings, there is less information on the distribution of boundary shear stress. Blodgett (1984) reports that because bridge piers decrease the efficiency of a river section, an increase in the mean velocity of flow takes place through the bridge. The ratio of maximum velocity to mean velocity was reported by Blodgett as increasing by 14 percent in a typical bridge opening.

The velocity distribution in channel bends has been measured and studied by a number of researchers. The equation developed by Rozovskii (1957) is widely used to estimate the magnitude of the traverse velocity component of bend flow. Richardson et al. (1987) in Highways in the River Environment, derive an equation for the longitudinal velocity over the width of a stream for a gentle bend of parabolic cross section. Measurements by Ippen et al. (1962) have been widely used as the basis for determining the boundary shear stress in bends in many design procedures (i.e., EM-1602, SCS TR-25, and

Anderson). Improved measurements on boundary shear stress in channel bends with alluvial material were made by Nouh and Townsend (1979) using a laser dopler anamometer.

When the channel boundary is free to move, sediment transport processes become important factors in the stability of the river reach. Sediment transport factors are usually referred to by the scale of the phenomena and include local scour, general aggradation/degradation and regional aggradation/degradation. The regime of the flow with sediment transport is also important, since bed forms occur in the channel and will cause displacement of the mean bed elevation. Sediment transport effects govern toedown requirements for channel protection and may lead to additional freeboard. Jones (1984) summarizes various local scour equations associated with bridge crossings. Methods for calculating general scour due to bridge openings are given by Richardson et al. (1987) in Highways in the River Environment. Computer models are also used to calculate general scour at bridge openings, several of which are discussed by Holly et al. (1984). A general design procedure for evaluating toedown and freeboard requirements in alluvial channels is given by Simons, Li & Associates, Inc., (1985), which assesses the cumulative effect of bedform height, local scour and general aggradation/degradation.

Posey (1974) conducted a series of tests to evaluate riprap scour protection for bridge piers. Circular and wall pier shapes were studied and in the case of the wall pier shape, the pier was both aligned to the flow and skewed 30 degrees. The piers were protected by graded layers of material, meeting Terzaghi's inverted filter criteria. The flume test was made with a live, sand-bed and during test flows, dunes were the dominant bed form. In selecting the maximum particle size, Posey made the rough estimate that the velocity at the side of the pier was about double the average approach velocity. To protect the area around the pier, the riprap was extended slightly further than the edges of the scour hole that formed without protection. It was found in the degrading conditions, that the protection layer bedded down without losing material at the edges, but some edge settlement was noted during the passage of dunes. Leaching of bed material through the protection did not occur, indicating the utility of the Terzaghi gradation. Posey recommended that piers be protected using a riprap blanket with an inverted filter gradation placed 1.5 to 2.5 pier diameters in all directions from the face of the pier. A chart for determining riprap size was developed where the size is a function of the shape of the pier, percentage of contraction and the Froude number of the approach flow. Protection of bridge piers was recommended for bridge sites that were experiencing settlement, not as a design procedure for new bridges.

Nece (1974) studied the effectiveness of the Washington State Department of Highways method of preventing scour at bridge piers using riprap. Seven bridge sites were studied and hydraulic data collected. However, all the sites studied were relatively new and had not been subjected to major flood flows.

2.4 Site Characteristics

The location of a bridge crossing can have a significant effect on the methods and extent of stabilization required. Bridges located in an adverse reach of the river such as a channel bend, or a severely braided channel, will encounter dynamic channel conditions. The objective of achieving a stable waterway through the bridge opening may run counter to the fluvial processes underway in the channel. Bridge crossings that are not correctly aligned with prevailing hydraulic conditions in a reach, can encounter severe scour and erosion problems. Blodgett (1986) collected data on the geometric properties of open channels which showed that the geometry of open channels follows a consistent pattern. Detailed measurements by Blodgett on a single channel reach (Pinole Creek at Pinole, California) showed that a channel section can vary significantly over time. This change in channel geometry results in a variation in hydraulic conditions at a structure over time. As Blodgett points out, a survey of a channel section at any given point in time, cannot be taken as providing an absolute definition of the geometric properties of the reach. Rather, it should be viewed as one sample from a population that varies over time.

References that present a general overview of bridge location requirements include: Guide to Bridge Hydraulics (Neill, 1972); "Hydraulic Analysis for the Location and Design of Bridges" (AASHTO, 1982); and, "Highway in the River Environment" (Richardson et al., 1987). These publications place an emphasis on channel response, scour protection, and channel training works.

They provide a guide to hydraulic design in fluvial systems with a particular emphasis on bridge waterways.

One particular aspect of bridge sites that has received increasing attention is the geotechnical aspects of bank erosion. Methods for evaluating the stability of bank slopes are presented in Design of Open Channels (SCS, 1977). Conditions causing slope failure are varied and no single procedure addresses all types of slides. Design of Open Channels addresses rotational slides, and translatory slides. Based on field inspection, Blodgett and McConaughy (1986) identified three types of slides that commonly occur in conjunction with riprap bank protection. Translational slide failures were associated with bank side-slopes that were overly steep; banks that had been undercut by bank degradation or scour; or the presence of excess hydrostatic pressure that reduces the internal frictional resistance of the slope. A modified slump failure is associated with riprap placed near the angle of repose, or loss of support provided by key stones in the riprap matrix resulting in downslope movement of the riprap. The slump slope failure is a rotational slide associated with the formation of fault planes due to nonhomogeneous base material with layers of impermeable material. Causes for slump type failure are: overly steep side-slopes, to the point where the gravitational forces exceed the forces along the friction plane, or excess overburden at the top of the slope.

2.5 River Response

River channels continually adjust to changes in water and sediment discharges in order to maintain a dynamic equilibrium. These adjustments involve changes in channel geometry over a substantial region of the river. These changes in channel form may have significant consequences at a bridge site. Therefore, an analysis of river morphology is necessary to understand the effect of potential changes in regime on geometry and channel pattern. The quasi-equilibrium channel geometry is usually related to slope, discharge and sediment properties (Lane, 1957; Leopold and Maddock, 1958; and Schumm, 1960). These relationships are not continuous and several thresholds have been shown to exist between river patterns (Schumm, 1974). The empirical relationship for river morphology and thresholds are based on laboratory and field data. Lane (1957) and Leopold and Wolman (1957) presented threshold channel slopes as a function of discharge (mean annual discharge or mean annual flood), separating meandering rivers from steeper braided rivers. Threshold conditions were observed in laboratory studies by Schumm and Khan (1972). The basic threshold for channel formation is the discharge at which bed material movement begins (Schumm, 1974).

Rivers may be classified according to channel pattern or type. The three major patterns have been identified as straight, meandering, and braided (Leopold and Wolman, 1957). Brice and Blodgett (1973) classified alluvial streams into four major types in order of increasing channel slope or bank full discharge, they are: equiwidth point-bar streams; wide bend point-bar streams, braided point-bar streams, and braided streams without point bars. Equiwidth point-bar streams are relatively narrow and deep; the width is not sensitive to changes in channel slope. The widths of the other stream types vary in direct relation to the slope and are sensitive to changes in slope.

Trent and Brown (1984) give a useful procedure for recognizing the potential channel instabilities in conjunction with the design of highways in river environments. They classify factors affecting river stability as natural or accelerated. Accelerated erosion typically results from man's activities in the river system. The procedure requires an understanding of geomorphic processes occurring within the watershed and an awareness of activities affecting river stability.

III. RIPRAP DESIGN REQUIREMENTS

Based on the review of riprap design practice in Arizona and a review of the literature on riprap research, the following design requirements have been found to be essential to conduct a complete design.

3.1 Riprap Properties

The use of the median size of a riprap gradation is being re-evaluated as the characteristic size describing riprap stability. Flume test conducted by the Corps of Engineers show the D_{30} size to be a more reliable predictor of riprap stability. The definition of a characteristic riprap size is key design criteria.

It is important to have a riprap gradation that provides an integrated mass of riprap protection, without voids or large areas of small particle sizes. At the same time, the gradation requirements must be feasible to produce from available quarry sources. The definition of a usable range of riprap gradations and a means of verifying this gradation in the field are important design requirements.

The thickness of a riprap blanket may compensate for small rock sizes. The thickness and gradation go hand-in-hand to produce a competent protection. The thickness is an important design requirement and interdependent with the characteristic size and material gradation.

Use of filter bedding or fabrics is basic design requirement. The Patagonia Case Study raised an interesting question on the performance of filter fabric. The literature search pointed out a similar concern by the Corps of Engineers in some of their field testing of filter fabric.

The durability of rock used as riprap is important both in the transportation and in-place performance of the protection. The Vanar Diversion Case Study, documents the reduction in riprap size during the shipment of riprap. This appears to be a fairly common occurrence and an effect that the designer should take into consideration.

Other design requirements for riprap characteristics which are better understood as to their effect on riprap stability include rock shape and density. It is important that the designers have information on the quality of the material produced by a quarry and some clear rules for evaluating the quality.

3.2 Hydraulic and Sediment Transport Characteristics

The incipient motion criteria for riprap particles is still undergoing fairly extensive research by the Corps of Engineers and Federal Highway Administration. These independent efforts (one based primarily on laboratory tests, and the other on field measurement) are producing similar results. These research programs have superceded most previous research and, therefore, should be the basis for riprap particle stability requirements.

The boundary shear stress is the force that must be resisted by riprap protection. Methods of estimating boundary shear stress are limited to relatively simple hydraulic conditions. The force placed on riprap protection in complex hydraulic conditions, such as channel bends, regions of accelerating/expanding flow, or where a local dissipation of energy occurs (piers and abutments), are more difficult to assess. The determination of boundary shear stress is a very important design requirement.

Degradation of channel beds was common to many of the case histories presented. The degradation problem appears to extend beyond local conditions created by the bridge and involves other activities such as sand and gravel mining, or development encroachment on the river. In developing riprap toedown requirements, degradation producing activities will need to be considered.

The use of riprap at bridge piers to control local scour is a common practice in Arizona and other states. The basis for the design of such protection appears to be quite limited. The procedure is a necessary and cost effective countermeasure for many bridges in degrading channels.

3.3 Site Characteristics

There is a need for the designer of riprap protection to recognize the degree of variation in channel conditions that can take place at a bridge site. As the Case Histories have pointed out, there can be significant variation in channel geometry, alignment and gradient at a site. Within reasonable limits, the designer must anticipate these changes and protect critical components of the site accordingly. The design requirements in this case extend beyond assessing riprap stability at the site into an assessment of river response.

A specific site consideration and design requirement pertaining to riprap is bank stability. An assessment of bank stability should be incorporated into riprap protection design. Detailed geotechnical analysis will not be necessary in most cases, but a qualitative assessment of bank stability should be a basic requirement.

3.4 River Response

An assessment of current river regime and threshold levels should be incorporated into design of riprap protection at a site. Man's activities in the river should be given particular attention. The use of checklists and geomorphic relationships can aid the designer in evaluating river response.

IV. OVERVIEW OF CASE HISTORIES

The development of a set of case histories, documenting riprap performance in Arizona, was the primary objective of Task One. While the focus of the study is on the use of riprap for protection of bridge sites, a comprehensive effort was made to gather case histories from any agency with either research or design experience. Federal, State, County and local agencies were contacted and interviewed over the telephone, and the following questions asked:

1. State the purpose of the study as follows:
   We are looking for installations of riprap channel protection, either for bank stabilization or protection of an in-stream structure such as a highway crossing, that have had documented flood flows. Do you know of these type of installations that have been built or are maintained by your agency?
2. If yes, can you tell me where they are located and the projects that the installations were constructed under?
3. Are design plans and calculations available for these projects?
4. Do you have data on flooding that occurred at these installations, or documentation on flood conditions that are on file in your office?
5. Does your agency use a specific methodology for designing riprap protection? If yes, get a manual reference or a copy of the design method.

Based on the initial telephone interview, office visits were scheduled to review in detail documentation of specific projects. Office visits were made to Pima County Department of Transportation, City of Tucson, Soil Conservation Service, and ADOT's headquarters. Recent floods in 1983 caused extensive damage to bridges in Tucson and Pima County. A report was prepared by the Pima County Department of Transportation and Flood Control District that catalogues the damages to public facilities and private property throughout the county. With the assistance of city and county personnel, bridge sites were identified where riprap, gabion, or rock and rail type bank protection had been used for protection of the structure. Construction plans were retrieved for sites at Swan Road, Rillito River Bridge; 22nd Street, Pantano Wash Bridge; Pantano Wash, North of Speedway Boulevard; and Ajo Way, Santa Cruz River Bridge. Because documentation of 1983 flood damages was extremely limited, none of these sites was selected as case histories for this study.

The Soil Conservation Service sited twelve project locations that might be used as case histories. After reviewing information on file at the SCS, two projects were selected as case histories:

Vanar Wash
Sonoita Creek at Patagonia

It was found that the SCS does not typically have the opportunity to conduct follow up evaluations on project performance. This occurs for a number of reasons, some of which are:

* Many SCS projects are designed for relatively frequent flooding conditions (25-year flood frequency). Major floods usually cause significant damage resulting in loss of the property, which results in removal of the property from the flood prone area. The SCS project is therefore not repaired since its function no longer exists.
* The SCS is not an emergency relief agency; and, therefore, they do not gather data on the immediate impacts of flooding.
* The SCS channel stabilization projects are design oriented, data collection on channel behavior or riprap performance is typically not an objective.

Also, the SCS has undertaken most of their major projects in response to recent major floods. As of yet, these projects have not received any significant floods since their completion. The case histories, that SCS has had an opportunity to prepare, are quite good, particularly the one for Vanar Wash. One drawback, to the use of SCS projects for this study, is that these projects are solely for bank stabilization and flood protection; neither case history has a bridge crossing in the project area.

The primary source of case histories was the bridge maintenance files of the Arizona Department of Transportation. Projects conducted by ADOT include original Construction at bridge sites, emergency repair projects after flood damage, and on-going projects for repair of scour damage to bridge sites. Because of extensive flooding in 1977, 1978, and 1983, a large number of bridges were damaged. Damage to bank protection also occurred, and at some sites a series of repair projects have been required. With the assistance of the staff of the Hydraulics Group at ADOT, bridge sites were selected from emergency repair projects. These included bridges at the following locations:

I-19, Santa Cruz River
I-19, Old Junction Wash
I-19, Tinaja Wash
I-19, Agua Fria Canyon
I-10, Rillito River
I-19, Esperanza Wash

The need to implement scour protection measures at bridge crossings in Arizona was identified in 1979. In a joint effort by ADOT and FHWA, a multidisciplinary team of hydraulic, foundation, and structural engineers was formed with the objective of identifying bridge sites with chronic scour problems. The Scour Team initially inspected twenty bridge sites in 1979, and this inspection subsequently resulted in thirteen construction projects at fourteen of the sites. Since initiation of the Scour Team, over one hundred inspections have been conducted resulting in construction projects at 70 sites. Construction funding of countermeasures is budgeted annually, and limits the number of projects that the Scour Team can undertake each year. Through 1985, about $7.0 million of scour-countermeasure projects were funded by ADOT. Funding on the order of $1.0 to $1.5 million per year for bridge countermeasure projects is anticipated by the Structures Section at ADOT.

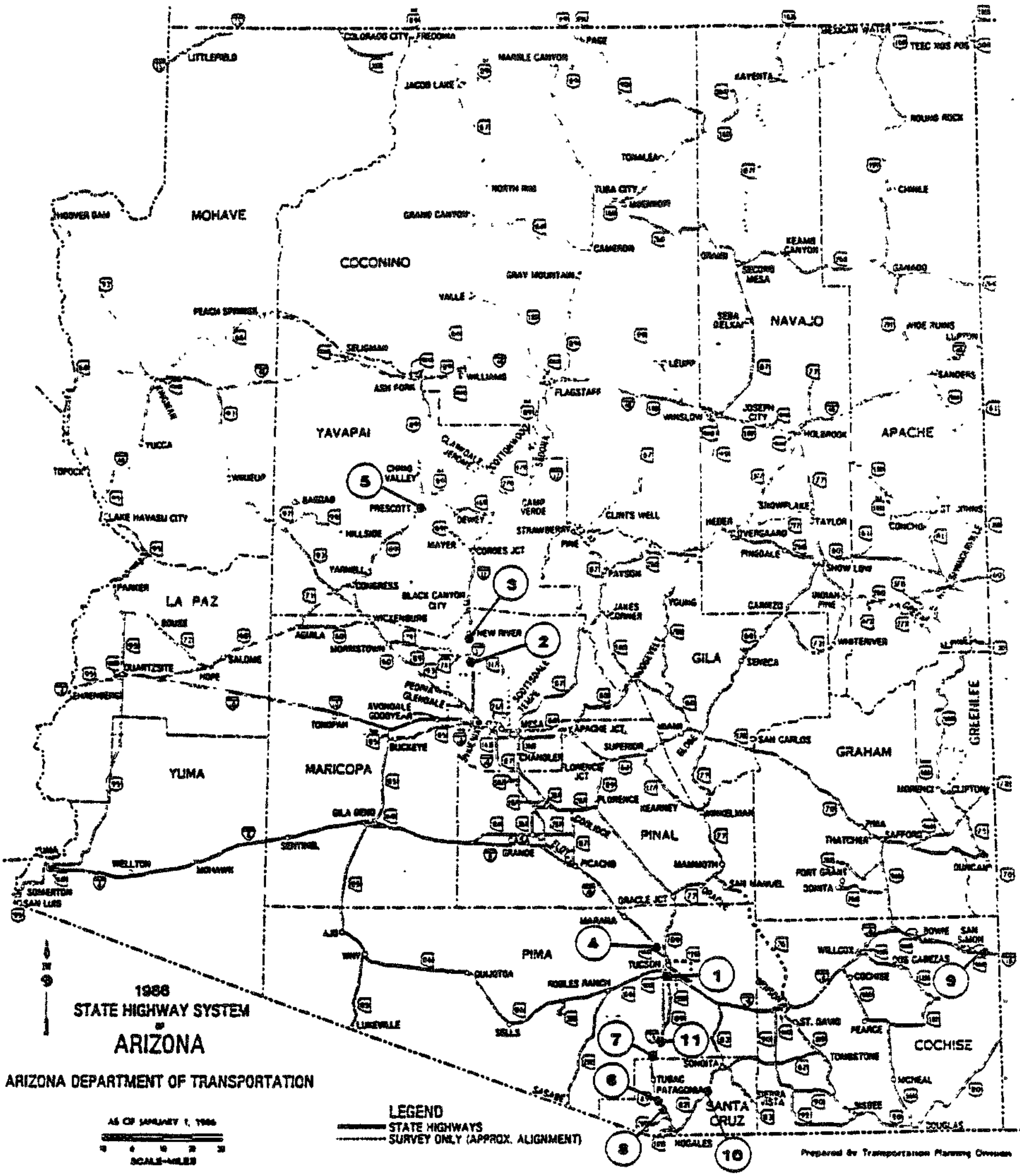
Case histories were selected from Scour Team Projects that involved riprap protection measures. Information was gathered on projects at the following locations:

I-17, Deadman Wash
I-17, New River
US 89, Granite Creek

V. CASE HISTORIES

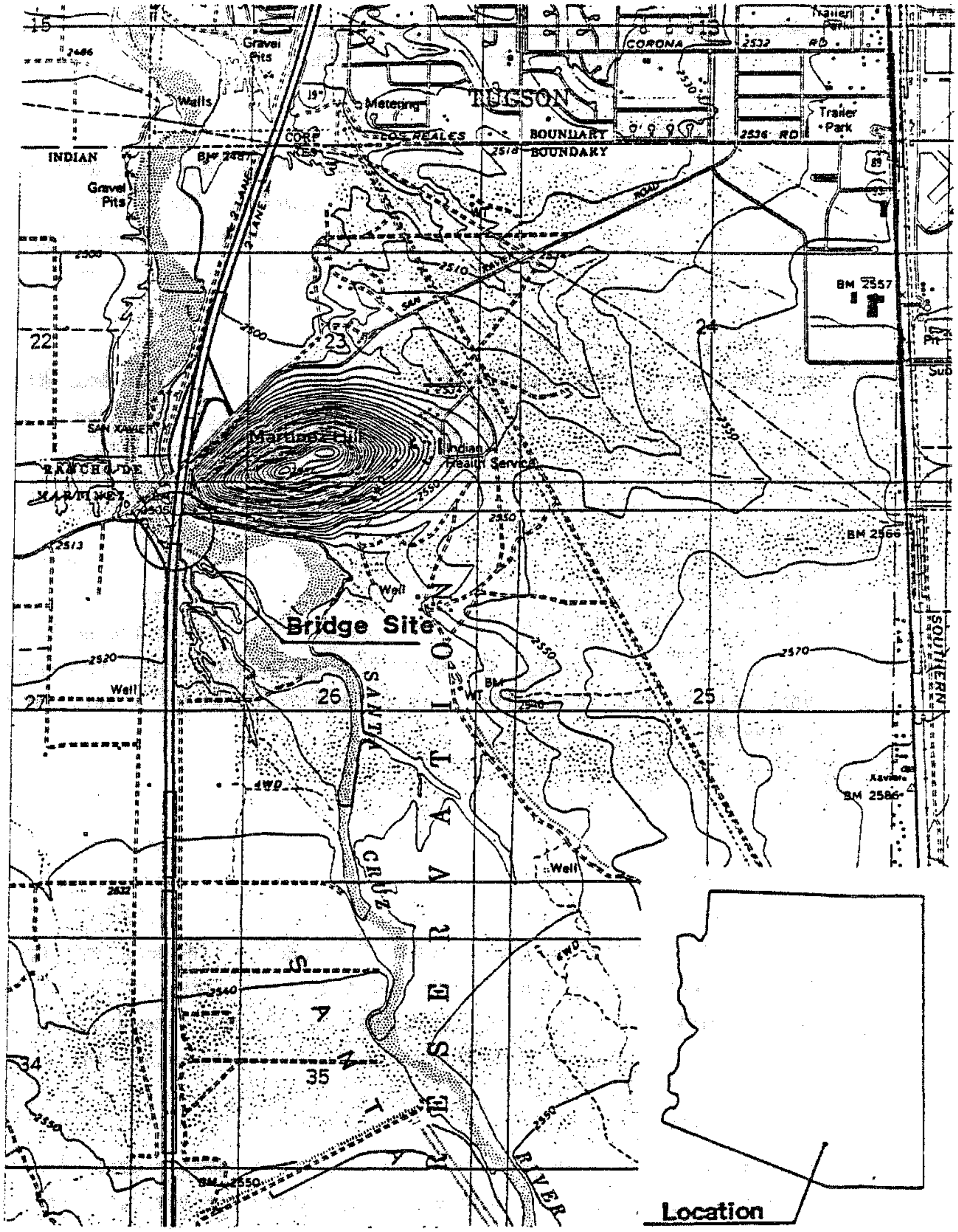
The general format used in this report to present case histories information is to describe the location and type of bridge crossing, give the hydrologic and hydraulic characteristics of the associated river reach, provide a general history of the site, and present a chronology of construction and repairs at the site. Information varies for each of the sites and with the source of information.

The locations of the various bridge sites that were included in this study are shown in Figure 5. The locations are geographically well distributed in Arizona and are typical of many bridge sites in Arizona. The drainage basin size varies significantly at each location from six-square miles at Old Junction Wash to 2070 square miles on the Santa Cruz River at the I-19 site.



SITE LOCATION MAP

Figure 5



CASE HISTORY NO. 1

Santa Cruz River
Route: I—19

Case History No. 1, I-19, Santa Cruz River Bridges

Location: Southwest of the City of Tucson

Type of Crossing:

Two bridges: Northbound is approximately 5160 feet long with four piers (5 spans). Southbound is approximately 410-feet long with three piers (4 spans). Bridge skew varies because the structures are located on a 0° 18-minute curve. Bridge skew approximates 45° for the most part.

Hydrologic characteristics:

Drainage area = 2070 square miles, which provides a maximum highway design flow of 45,000 cfs. This is the maximum discharge of record, October 1983. Water surface elevation was estimated at 2495.82 feet.

Hydraulic characteristics:

The average depth of flow (D_{max}) = 12.0 feet with an average velocity approximating 11.7 ft/sec. The location of these structures are unique in that the Santa Cruz River, in its attempt to penetrate the buttes, dissipates energy prior to entering the crossing. This is visible from large meanders upstream of the structures. This location creates a very unstable local regime for any type of crossing.

Crossing history: (1967-1977)

Flooding of the Santa Cruz River has caused severe damage to the bank protection at the project site since the 1967 construction. The first observed damage occurred in February 1968 after a storm in December 1967 during the original construction. The sheet piling was repaired in 1968 by placing concrete into the scour holes on both sides of the piling. During these years, hydraulic and structural conditions seemed to be satisfactory except for minor scour and channel degradation occurring near the south bank.

1977: (USGS Q = 22,000 cfs)

October 9 and 10 provided the Santa Cruz with another major flood event that dramatically changed the channel configuration and the bank protection at the south abutment of both structures. The dike along the east side of the south abutment was damaged by water which broke through the gullies from the meander upstream, came across country and flooded the area behind the dike. The dike was overtopped which allowed the undermining of the riprap and filter blanket. The riprap was washed away beginning next to the sheet piling at the northbound abutment and extending 160 feet east.

The cyclopean concrete poured into the scour area in 1968 effectively sealed the sheet piling from piping action in this area. The embankment fill was now lost beyond this prior treatment downstream under the southbound bridge for a distance of approximately 150 feet. Material lost behind the sheet piling extended back six to seven feet and down to the present stream elevation.

Wired riprap along the top of the sheet piling collapsed into the void behind the piles and had totally disappeared with only traces of wire mesh hanging from the top of the piling.

At that time, after the flood, the streambed had degraded about five feet under the northbound and southbound I-19 bridges. There had also been a great loss of river bank material on the north upstream side. The nine-foot bank was eroded completely back to harder material and had encroached within 20 feet of the sheet piling below the north abutments. The same type of river bank erosion occurred on the south side of the channel between the mainline and the Mission Road ramp bridge.

1978:
Another storm caused large runoff in the Santa Cruz River. Channel changes upstream put the force of the flow against the north bank protection and caused failure of sheet piling and rock protection. Emergency measures were taken to haul rock to reinforce the area under stress and prevent fill erosion.

After the damage, extensive rebuilding of the entire bank protection was accomplished by removing the sheet piling and replacing it with dumped rock riprap. This installation was monitored closely during each runoff. Each runoff caused settlement or realignment of the loose rock.

1983:

On February 4, 1983, a major storm event caused erosion of the northeast corner of the bank protection. Emergency measures were taken to haul heavy rubble fill to protect the abutment fill. The peak flow has been estimated at about 5000 cfs by the USGS.

The most severe damage was located at the northeast corner of the bridge site, upstream of the north abutment of the northbound bridge. The angle of flow was directed at this corner and as a result about 150 feet of the existing dumped riprap section was lost. Other site damage was minor, with some minor scour in the vicinity of the mainline bridge piers at the south bank. It was recommended that a spur dike with plating replace the dumped riprap.

* Missing October 1983 flood data.

Riprap design: 1978

ADOT designed the riprap required to protect the structures using Highway Research Board Report 108. With this design, ADOT determined a D_{50} = 24 with D_{max} = 36" would work best. There ended up being a difference in opinion with the FHWA. The FHWA design using HEC-15 determined D_{50} = 12". The ADOT hydraulic engineers would not agree with FHWA as shown in the structures hydraulic report due to discrepancy in stream velocity estimates. The specified riprap gradation was:

Percent Passing	Rock Size	
100	20"	D_{50} = 12"
35-55	14"	D_{max} = 18"
5-20	7"	
0-5	4"	

1979:

In a meeting between FHWA and ADOT Structures Section, the consensus was that the design procedures presented in HEC-15 would be used without modification. The following design criteria was specified:

1. Angular rock will be specified (Sec. 612.02, pg 115 of HEC-15).
2. Minimum weight of stone will be 155 lbs/cu ft (Sec. 612.02, pg 115 of HEC-15).

3. Velocity of flow V used in Chart 33, page 62 of HEC-15 will be the average velocity of flow, (Q/A).

1984:

In speaking with ADOT hydraulic design team leaders, it was found that the Highway Department has no consistent method of design for dumped riprap. One squad is using HRB Report 108, while another squad is using Laursen's Method from the 1963 proceedings of ASCE. The redesigned dumped riprap gradation using Laursen's method requires:

Percent Passing	Rock Size	
100	27"	D_{50} = 18"
40-60	20"	Minimum S.G. = 2.40
20-40	15"	rock = 150 lb/cu ft
0-5	6"	

Figure 6. Santa Cruz River topography at I-19 bridge after October 1983 flood. Scale 1"=133' (ADOT Hydraulics Group)

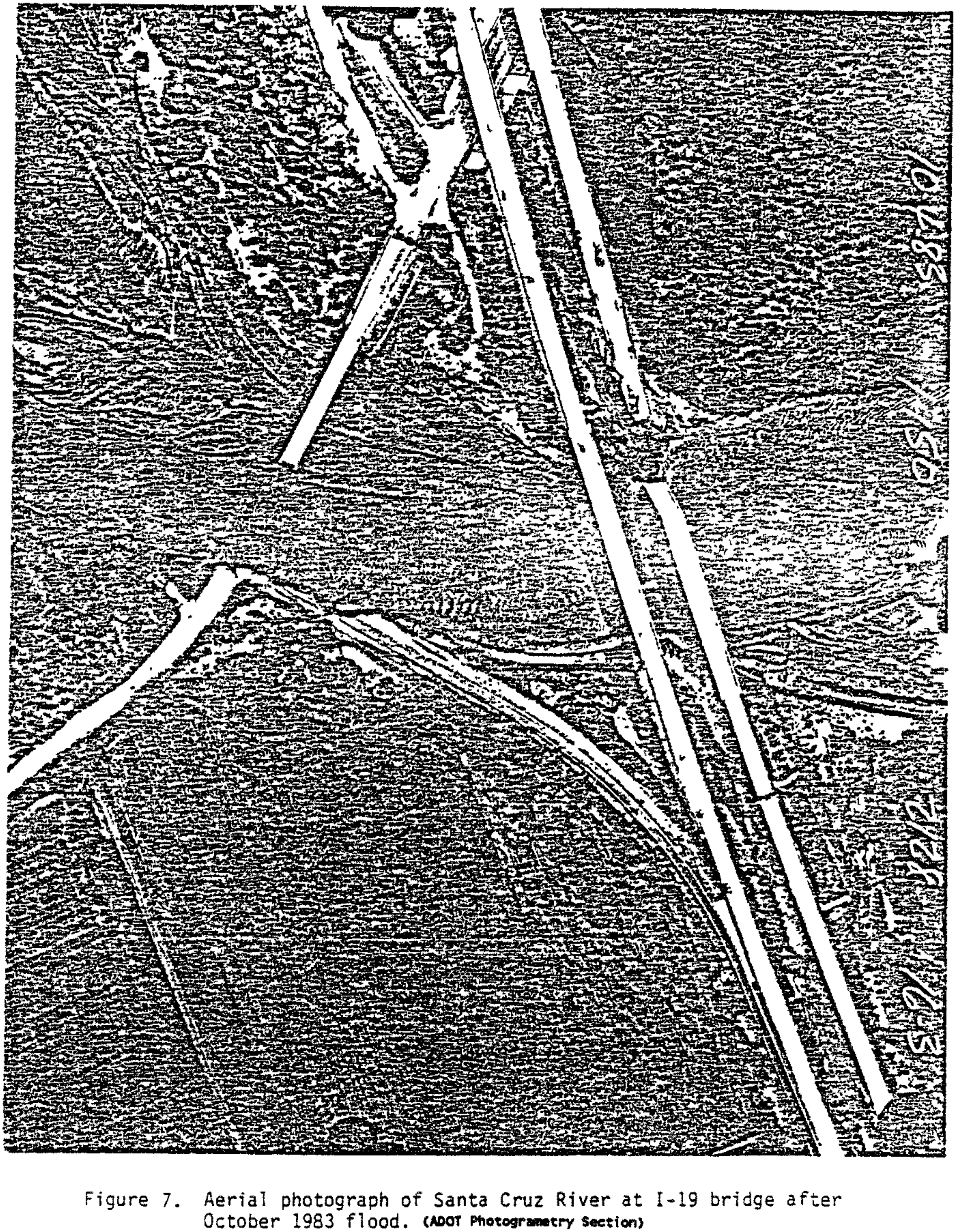
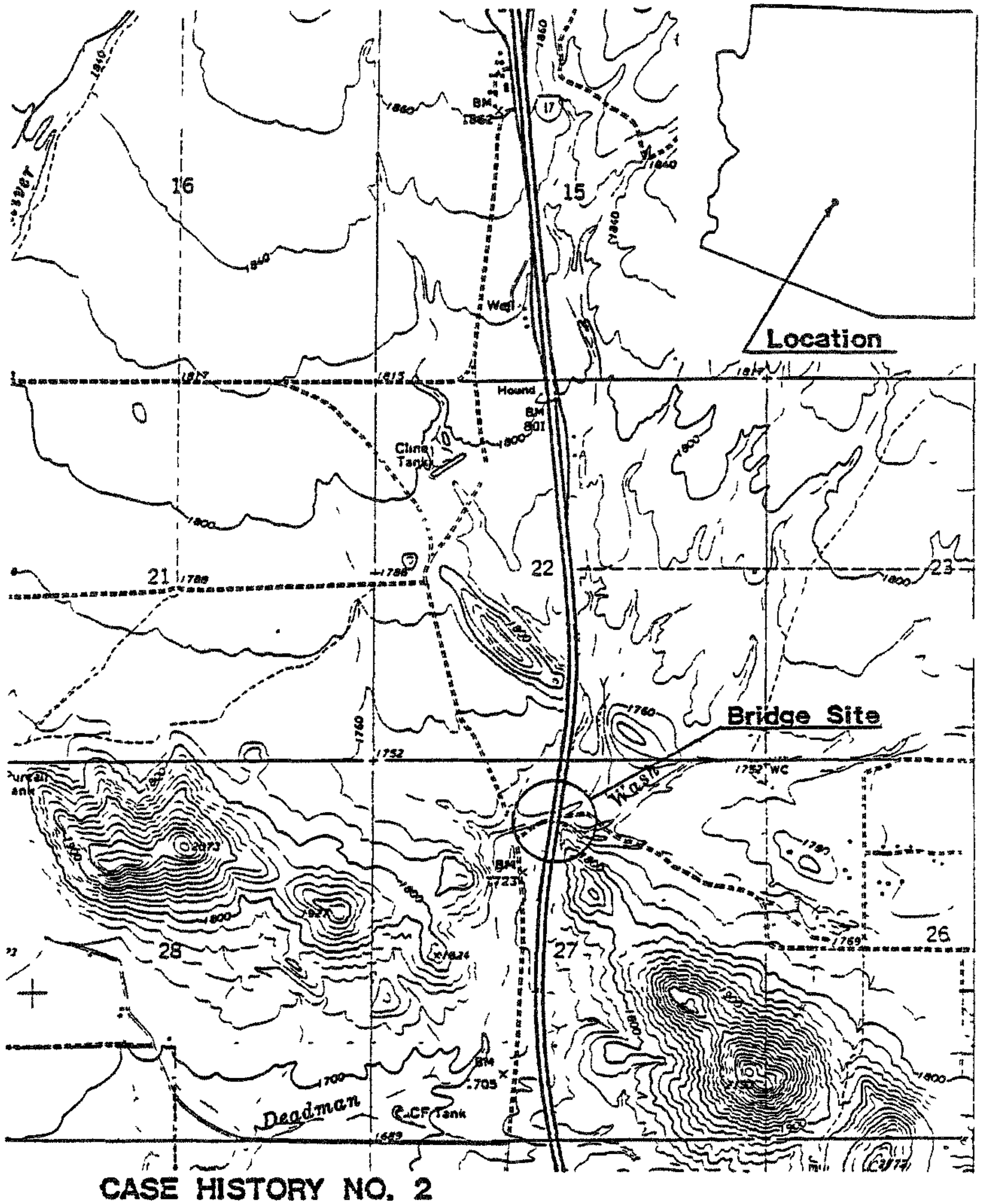


Figure 7. Aerial photograph of Santa Cruz River at I-19 bridge after October 1983 flood. (ADOT Photogrammetry Section)



CASE HISTORY NO. 2
Deadman Wash
Route: I—17

Case History No. 2, I-17, Dead Man Wash Bridge

Location: 27 miles north of Phoenix

Type of crossing:

Two bridges (northbound and southbound). Each bridge is 130 feet long, with three piers (4 spans) at a 0° skew to the bridge.

Hydrologic characteristics:

Drainage area = 12.4 square miles which provides a 50-year highway design flow = 8,800 cfs.

Hydraulic characteristics:

The average depth of flow (Q_{50}) = 2.9 feet, with a Froude No. (Q_{50}) = 1.1. The northbound bridge is located at the confluence of two major washes. The approach channel is skewed at least 15-20° with the piers at the northbound bridge due to the channel confluence. The southbound bridge has no skew and therefore is impacted by water at a 20° deflection caused by the northbound bridge.

General history:

As of 1979, the streambed had not changed significantly under either bridge since original construction and there had been no hydraulic related repairs done since original construction.

Crossing history: 1948

The original bridge was constructed (future southbound). A channel was constructed upstream for 400 feet and downstream for 200 feet. This consisted of dikes and embankments made of the material from the channel excavation. No riprap was shown.

1959:

December, a flood was recorded Q = 1850 cfs.

1965:

The second bridge is constructed (northbound).

1978:

March, a flood was recorded Q = 1400 cfs.

1979:

First report of channel condition. Scour was detected to within two feet of the top of pier #1 and three feet of the top of pier #2. The approach flow was split.

1979:

A new riprap invert and embankment is constructed. Riprap characteristics according to design criteria: The rocks are to be angular and minimum weight to be 155 pounds per cubic foot. The thickness of the mattress and toedown is three feet. The slope of the toedown is 3:1 and its length is 9.5 feet. The rock riprap gradation:

Percent passing	Size
100	36 inches
35-55	22 inches
5-20	10 inches
0-5	6 inches

Site characteristics:

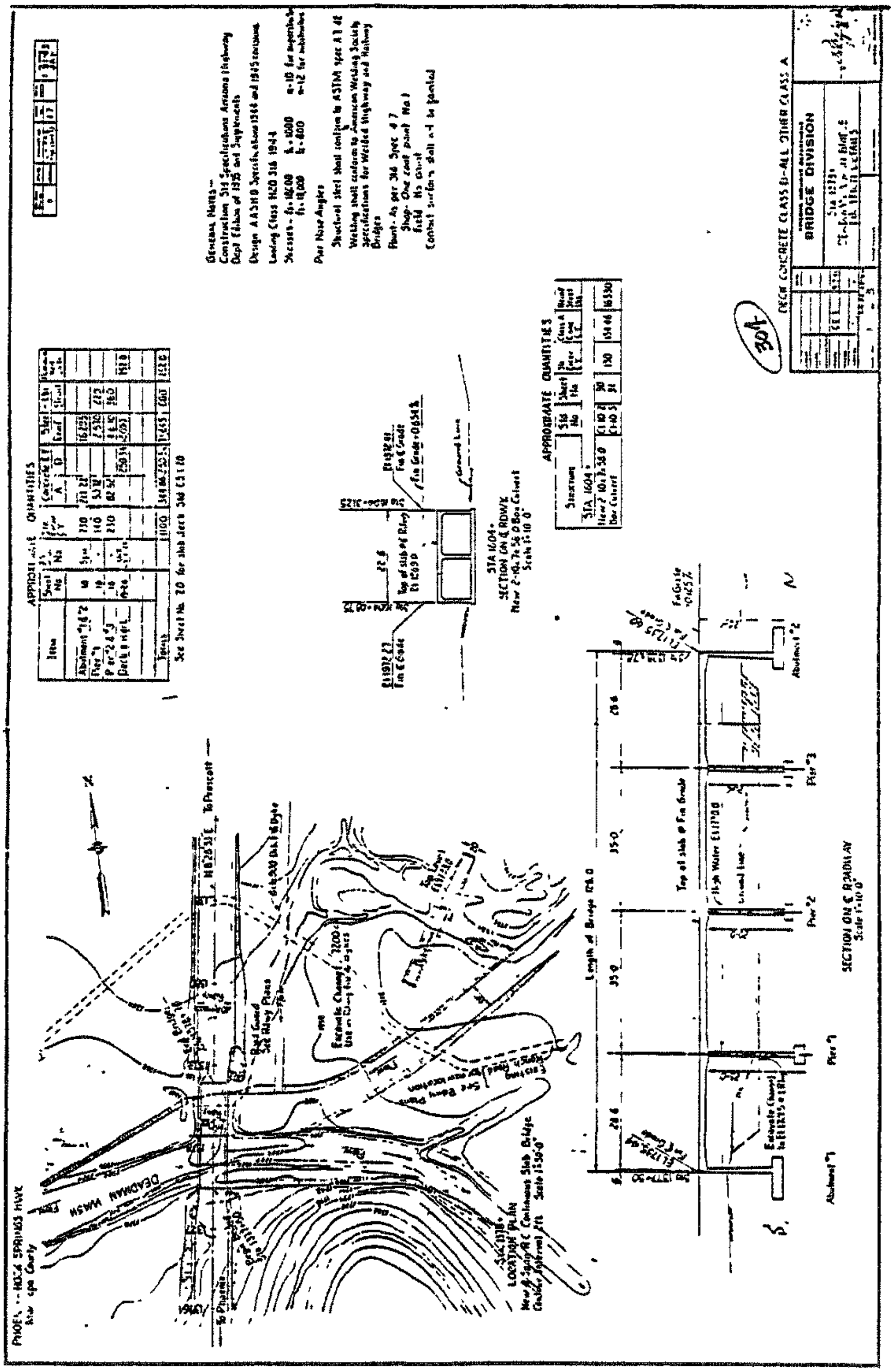
The side-slope of the bank is according to standard C-2.01 and the channel bottom was not protected.

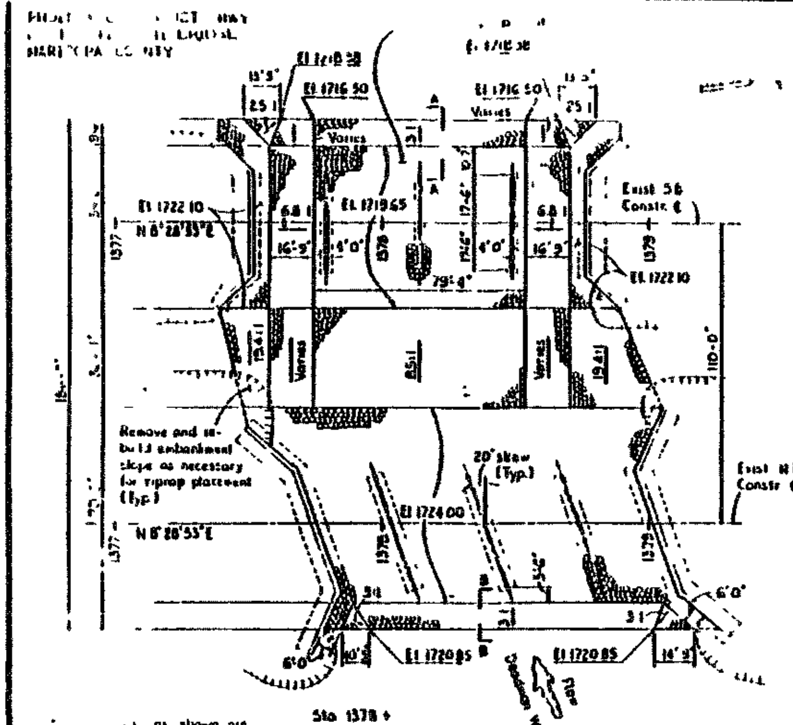
1980:

February, by looking at photos, there appears to be a recent high water event that scoured a one-to-three-foot deep area under the northbound bridge at the confluence point.

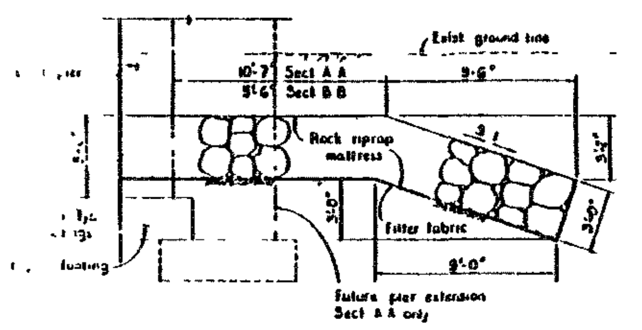
1982-1986:

The inspection reports state that there has been little flow and little alteration to the riprap mattress.

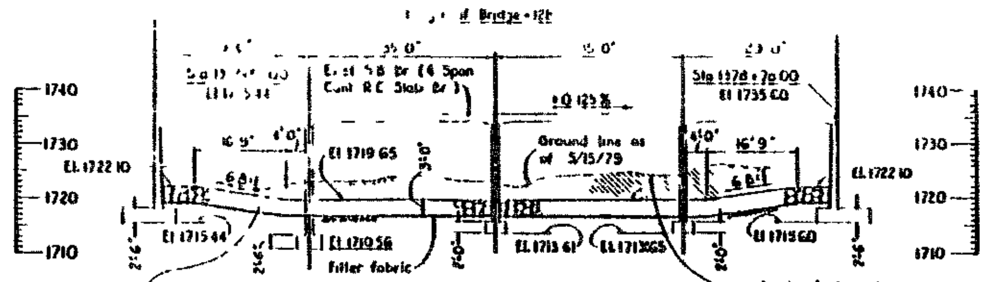

DEADMAN WASH
LOCATION PLAN
To Phoenix
To Prescott
SECTION ON ℄ ROADWAY
APPROXIMATE QUANTITIES
Abutment #1
Pier #1
Pier #2
Pier #3
Abutment #2
BRIDGE DIVISION
DECK CONCRETE CLASS B—ALL OTHER CLASS A



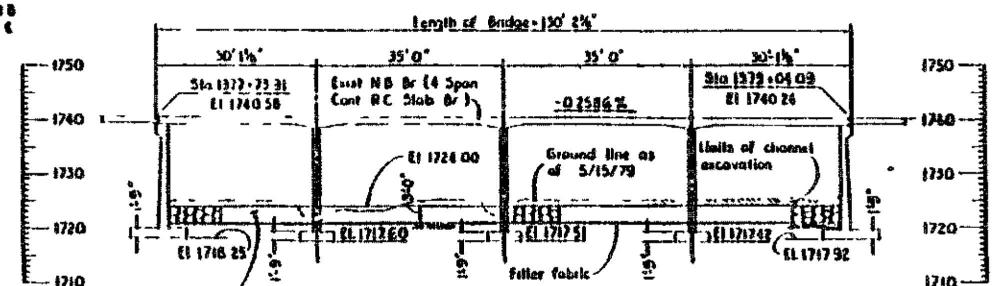
Sta 1378 +
LOCATION PLAN
Scale: 1"=20'0"



SECTION A A / B B
Scale: ½"=1'0"



SECTION ALONG EXIST. SB CONSTR. ℄
Scale: 1"=10'0"



SECTION ALONG EXIST. NB CONSTR. ℄
Scale: 1"=10'0"

APPROXIMATE QUANTITIES	
Channel Excavation	2600 C Y
Rock riprap	2830 C Y

ROCK RIPRAP GRADATION	
% Passing	Size
100	36"
35-55	22"
5 20	10"
0 5	6"

Note Rocks to be angular and minimum weight to be 155 lbs/ft³

NOTES:

Construction - Std. Specs Arizona Highway Department Edition of 1969, revised to date

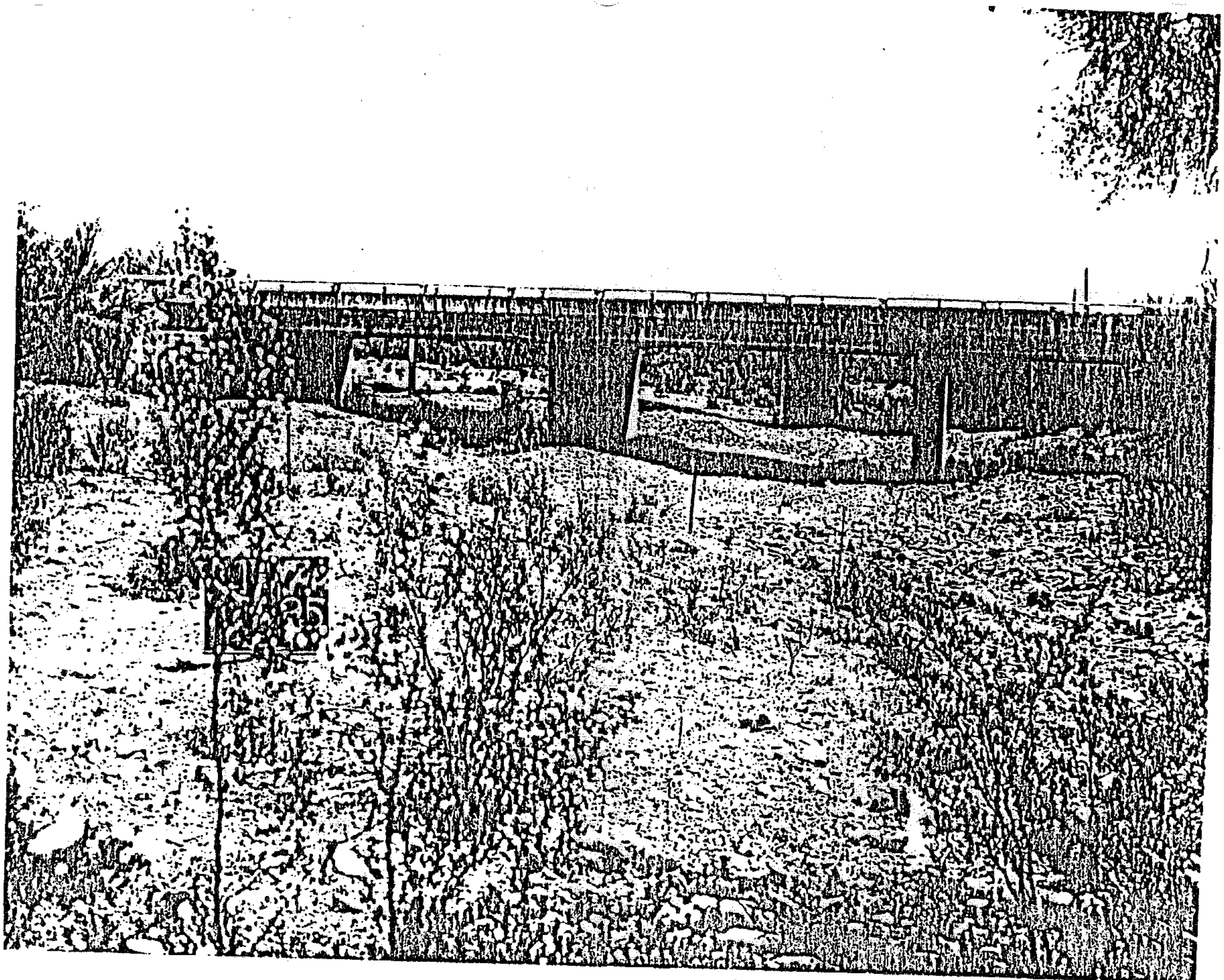
All stations and elevations shown indicate existing data on "As built" plans and do not necessarily correspond to structure conditions now existing

Excavation material shall be disposed of at the site under the direction of the Engineer

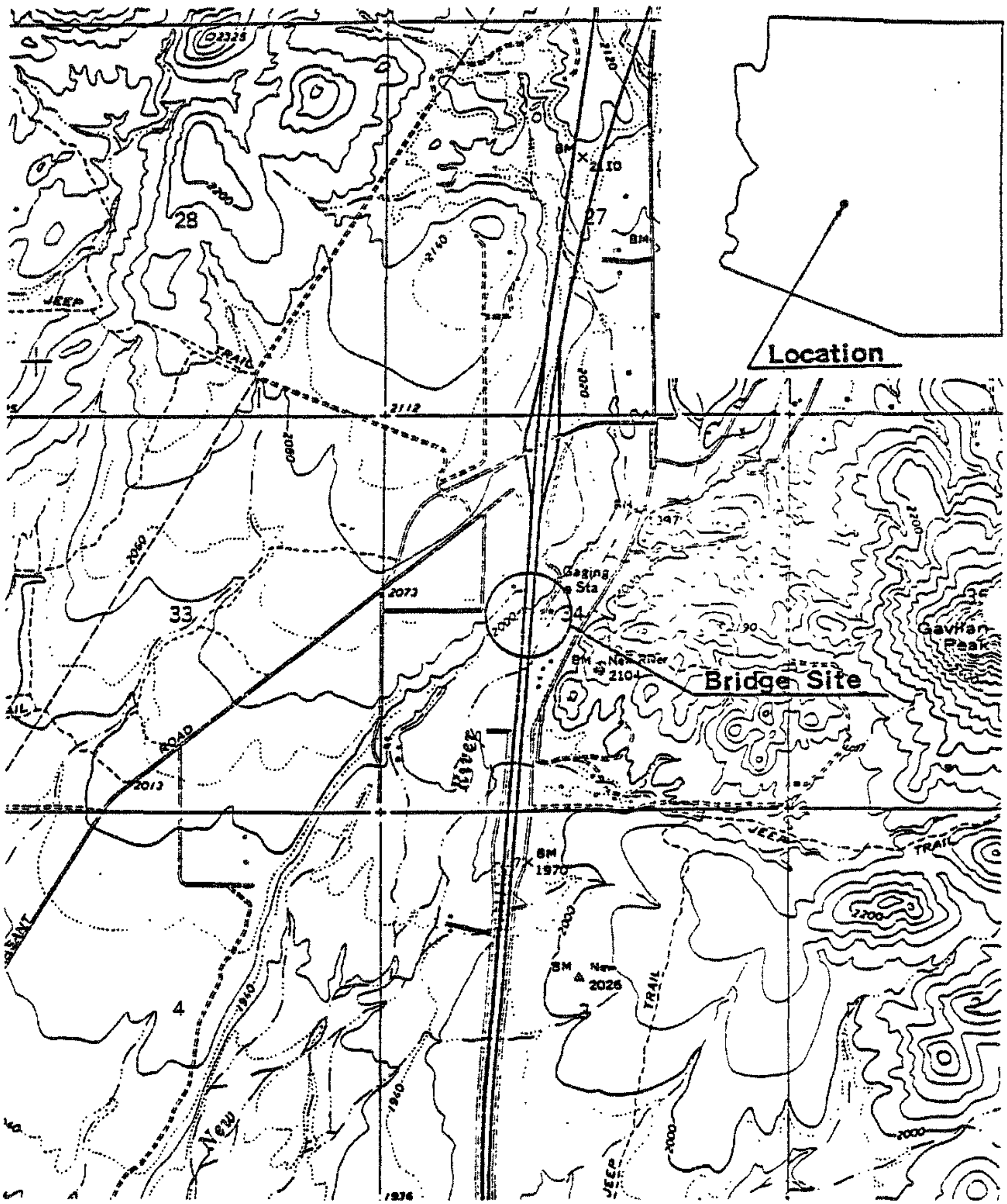
AS BUILT

SITE NO 2
JOINT FHWA/ADOT
BRIDGE INSPECTION
JUNE 1979

ARIZONA DEPARTMENT OF TRANSPORTATION
STRUCTURES SECTION
Sta 1378 + SB & NB
DEADMAN WASH BRIDGES
SCOUR PROTECTION DETAILS
E 17-1-393



(ADOT Bridge Inspection Files)



CASE HISTORY NO. 3
New River
Route: I—17

Case History No. 3, I-17, New River Bridge

Location: 31 miles north of Phoenix

Type of crossing:

Two bridges, northbound and southbound. Each bridge is 347 feet long, with eight piers (9 spans) at a 30° skew to the bridge.

Hydrologic characteristics:

Drainage area = 82 square miles which provides a 50-year highway design flow = 17,300 cfs. USGS Gauge 5138 is close to the site.

Hydraulic characteristics:

The average depth of flow = 5.4 feet with a Froude No. (19,500 cfs) = 1.1. The bridge is over a meandering channel with abutment 1 (south end) on the outside of a severe bend (900 foot radius). The piers are skewed 20°-40° to the flow. A concrete deflector wall upstream of abutment 1 has apparently mitigated the angle of attack at abutment 1 and piers 1 and 2.

General history:

The stream bed has not changed significantly under either bridge since 1971. During construction, a portion of the riprap washed out at the south embankment of the northbound bridge. Additional riprap and an upstream deflector wall were added by change order. In 1970, a portion of the upstream north bank riprap had to be repaired because of another washout.

Crossing history: 1967

December, a flood was recorded Q = 12,000 cfs.

1968:

The two bridges were constructed. The channel was improved for about 600 feet upstream and 450 feet downstream of the bridges by constructing a riprap guide bank for the upstream section and excavating a channel downstream. The excavated channel bottom width was about 220 feet to 250 feet.

Riprap characteristics:
The dikes were built from existing material within the channel bed and from existing borrow pit areas. Boulders with a minimum size of 50 inches were placed three feet thick on a 1.5:1 slope with a four-foot vertical toedown for a 175 foot length under both abutments. Riprap with a 15-inch minimum-size boulder was placed 1.5 feet deep on 1.5:1 slope with a four-foot vertical toedown in the upstream section of the channel. The center line of the channel had a 900 foot radius of curvature. The inside bank (north side) was 290 feet long and the outside bank (the north side) was 610 feet long. The slope of the upstream channel was 0.75% and about nine feet deep. This channel bed material is composed of sand/gravel. The downstream channel improvement was excavated from the existing bed on a 1.5:1 slope.

1970:
September, a flood was recorded Q = 19,500 cfs.

1971:
First bridge inspection report showed degradation of almost four feet under spans 2 and 3 near the south end of the structure.

1978:
January, bridge inspection report showed no change.

1978:
March, a flood was recorded at Q = 17,900 cfs, which was approximately the design discharge.

1978:
June, loose riprap had been deposited along both banks up to and in front of the end piers. The observed high water marks were four feet below the soffit.

1979:
March, a flood was recorded at Q = 4,600 cfs. The bank protection (dumped rock) failed between the upstream deflection wall and a point downstream of

the southbound structure. Local scour was evident at piers 3, 4, and 5 of the northbound structure. To protect the bridge piers, a riprap invert and embankment was constructed. The riprap was constructed along the outside of the bend and under spans 1-5 and half of span 6. It was built three feet thick of type "A" riprap with a ten-foot toedown at a 2:1 slope. The riprap embankment was constructed for a distance of 300 feet along the south bank and is 5.5 feet thick of type "A" riprap at a 2:1 slope. All riprap was placed on filter fabric. Rocks were specified as angular and minimum weight to be 155 pounds per square foot. The specified riprap gradation was:

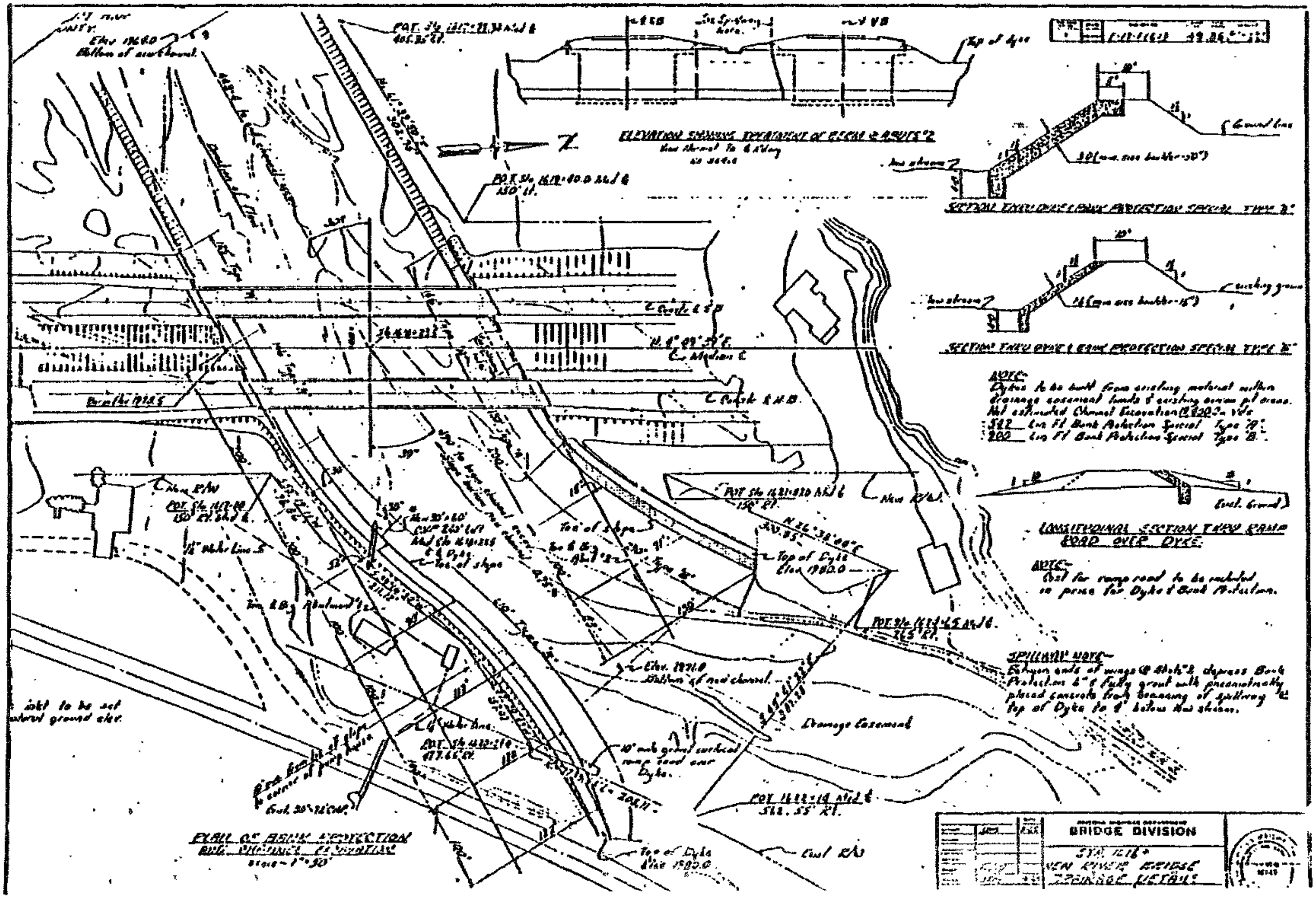
Percent passing	Type "A"	Type "B"
100	66 inches	36 inches
35-55	38 inches	22 inches
5-20	18 inches	10 inches
0-5	12 inches	6 inches

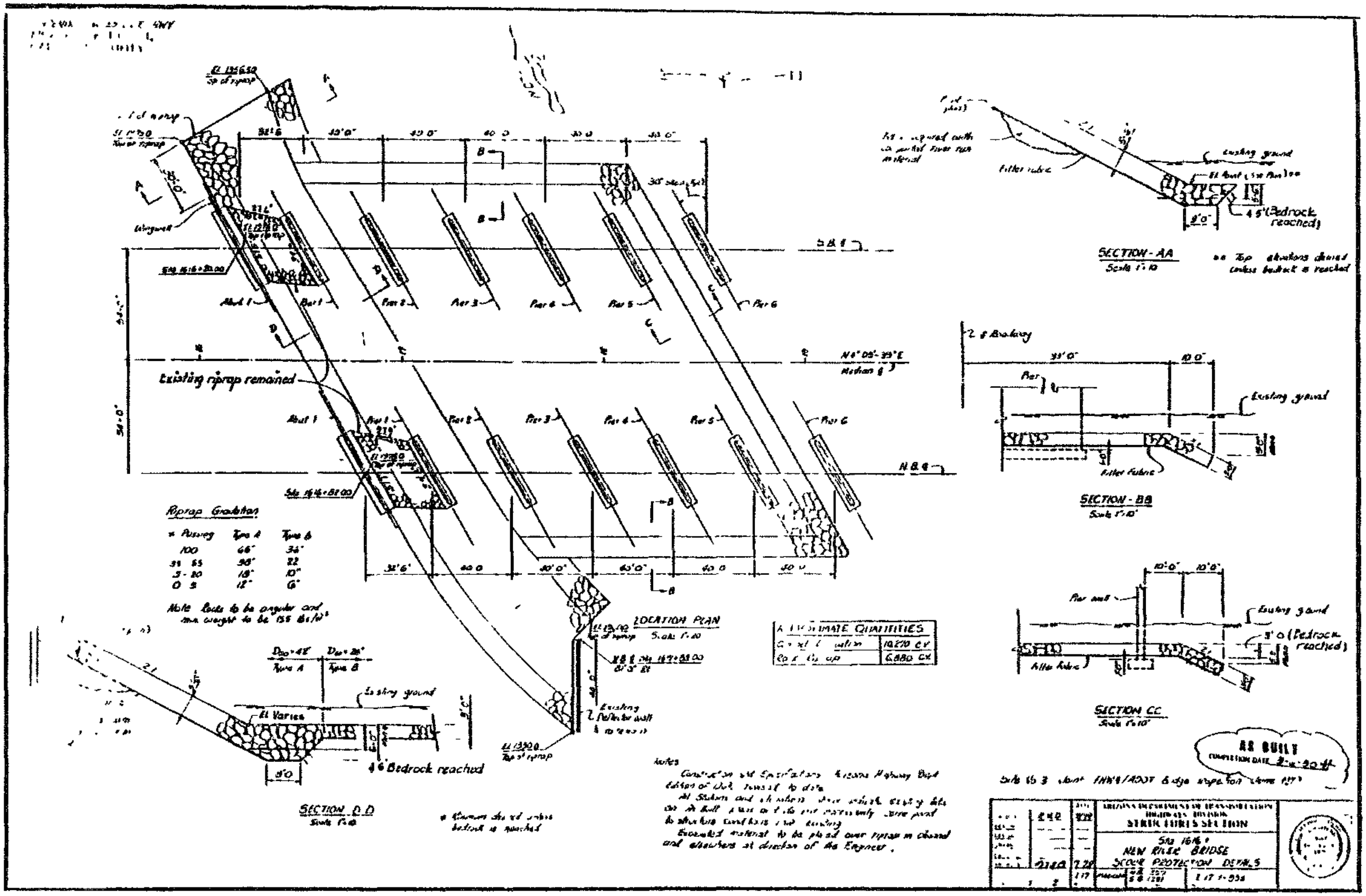
1980:

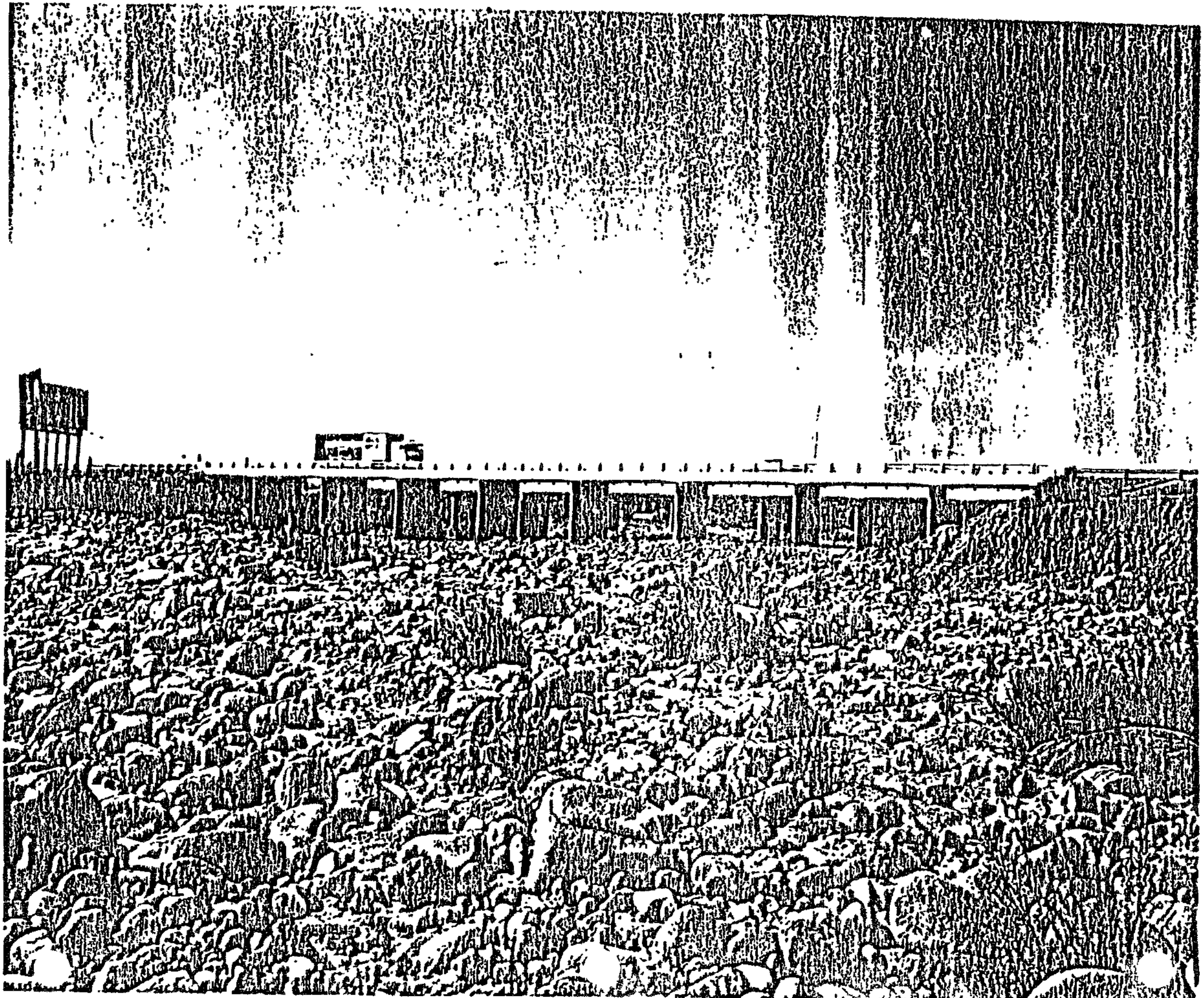
February, a moderate flood occurred. From the photos, pier 2 on the northbound bridge was at the three foot water depth mark. Witnesses observed high water at span 1 at the southend of the southbound bridge, debris was shown at a high water mark 7.5 feet below deck. Bank protection along south bank held well, no apparent scour at toe. Most of water flowed under spans 2, 3, and 4, some under 5; water flowed off the apron between the bridges under span 5 into span 4.

1984:

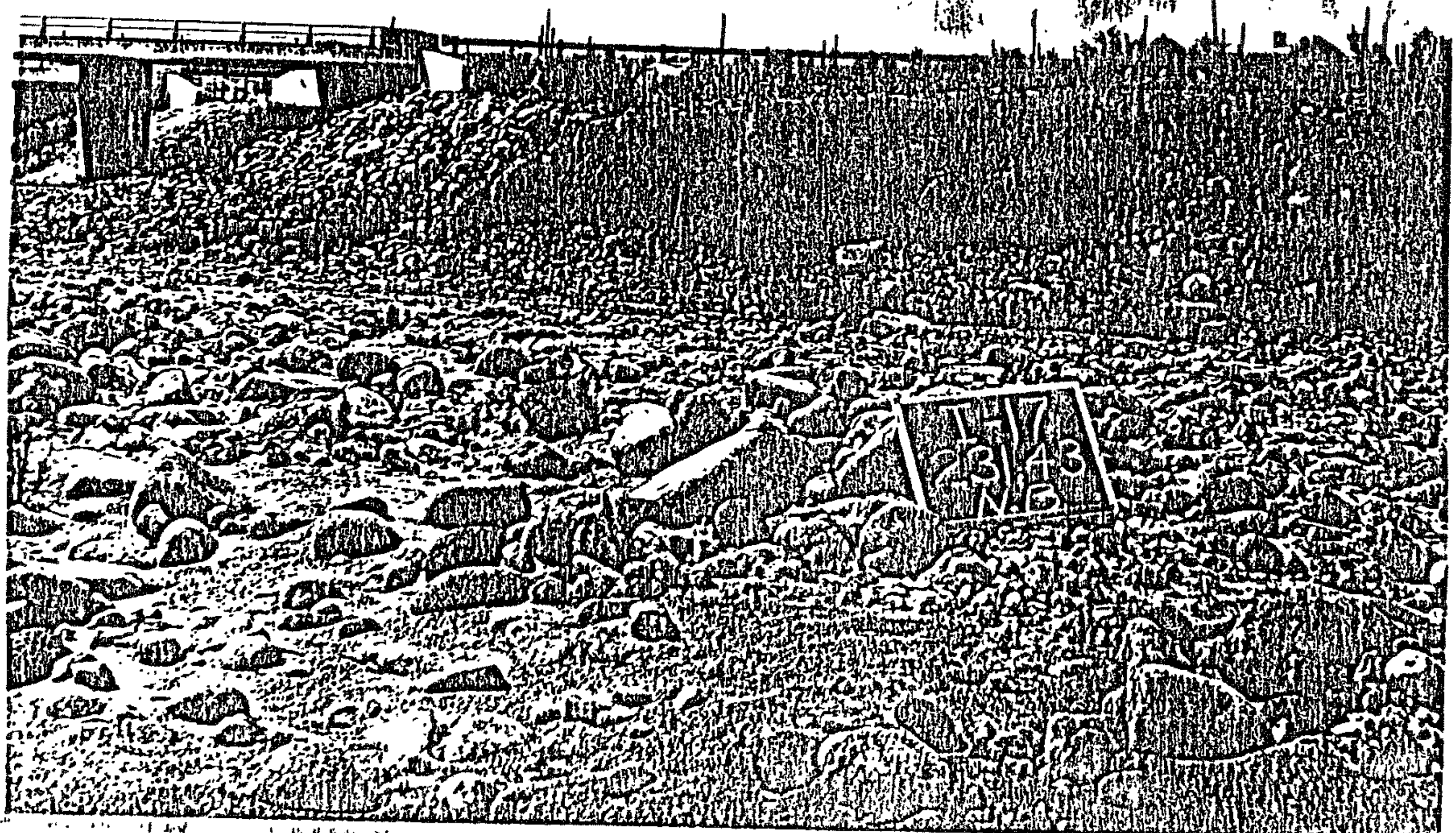
Bridge inspection report says the channel has remained relatively stable since last inspection; minor loss of material at piers 4, 5, and 6 is no problem at this time.


BRIDGE DIVISION
NOTE
Drainage Easement
Top of Dyke
Top of slope
LONGITUDINAL SECTION THRU RAMP ROAD OVER DYKE.
SPILLWAY NOTE

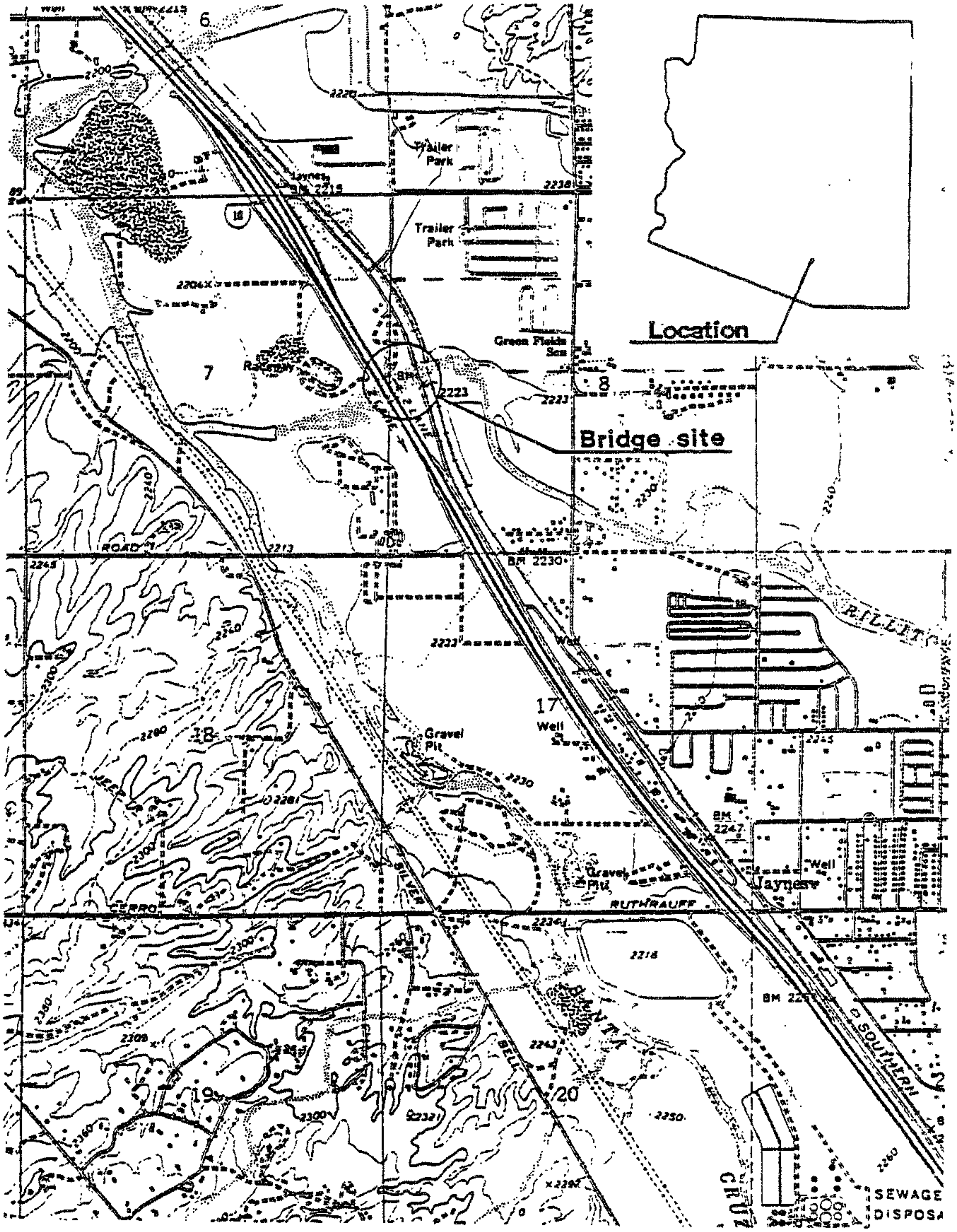

Existing riprap remained
Abut 1
Pier 1
Pier 2
Pier 3
Pier 4
Pier 5
Pier 6
LOCATION PLAN
SECTION - AA
4.5' (Bedrock reached)
SECTION - BB
SECTION CC
3' 0 (Bedrock reached)
SECTION D D
4.6' Bedrock reached
El Varies
Existing ground
Filter fabric
Riprap Gradation
Note Rocks to be angular and min. weight to be 155 lbs/ft³
APPROXIMATE QUANTITIES
10270 CY
6880 CY
AS BUILT
COMPLETION DATE
STRUCTURES SECTION
NEW RIVER BRIDGE
SCOUR PROTECTION DETAILS



(ADOT Bridge Inspection Files)



(ADOT Bridge Inspection Files)



CASE HISTORY NO. 4

Rillito Creek

Route: I-10

Case History No. 4, I-10, Rillito Creek Bridge

Location: 10 miles north of Tucson

Type of crossing:

Two bridges, northbound and southbound. Each bridge is 342'6" with ten piers (11 spans) and no obvious skew. Most spans are 32 feet long and constructed as a reinforced concrete continuous slab bridge.

Hydrologic characteristics:

Drainage area = 934 square miles, which provides a Q_{100} = 24,000 cfs (USGS Small D.A. Study, Sept. 1978). Pima County Flood Control District estimates the Q_{100} = 31,000 cfs.

Hydraulic characteristics:

The average depth of flow = 10.8 feet with an average velocity of 11.4 ft/sec. The river has been unstable with much sinuosity in the vicinity of the site, (see aerial photos). When these bridges were constructed, I-10 bridges, railroad bridge and east frontage road bridge, they were 500 feet downstream from a bend. In the past few years, the meander upstream has formed an active bend adjacent to the railroad bridge. There has also been suggestions that upstream mining activities have attributed to the rivers instability.

Crossing history: 1978 (Q = 16,000 cfs, Q_{20} per USGS)

The main flow, originally in the braided condition, concentrated into the road area under the bridges and was forced to make approximately a 90° turn. This change in river course caused severe scour at the north bank and caused deposition at the south bank during this December 18, 1978 storm event. The following is field notes from the bridge inspection crew:

East frontage road:

- total loss of dumped riprap downstream of bridge.
- high water mark at bottom of girders.
- channel degradation: toe of dumped riprap is now four feet above the low channel.
- approximately 50 feet of the north approach washed out.

- dumped riprap around north abutment is gone, as well as about 100 feet downstream.

Northbound and southbound bridges:

- heavy debris on all piers.
- pier pile caps visible.
- no apparent bank damage on either north or south sides.
- high water at four feet below bottom slab.

In a special inspection of the structure after the flood event, it was also noted that the existing Type A dike and bank had eroded away for the total length (415 LF ±) between the east frontage road and the westbound bridge.

1983:

As a result of the October 1, 1983 flood, the channel degraded three to four feet under the first seven spans. No apparent damage was caused to the railbank but the question was raised regarding depth of scour in these areas and whether adequate cover was present. The only other mention is of debris on the railbank.

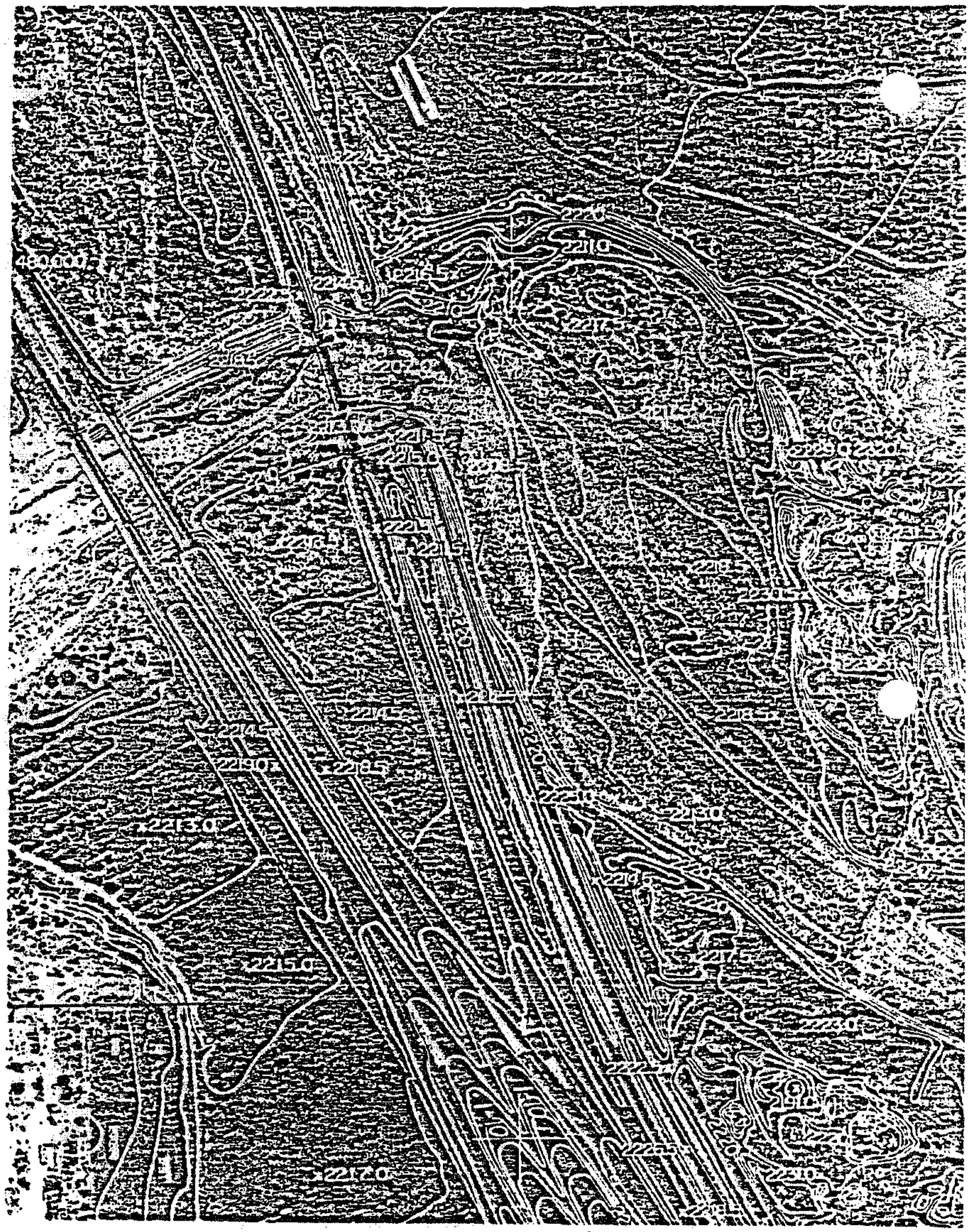
Riprap design: 1980

The method of design chosen by ADOT was HEC-15. The specified gradation was:

Percent Passing	Rock Size	
100	36"	
70-85	24"	D_{50} = 18"
40-50	18"	
30-40	12"	
8-15	6"	

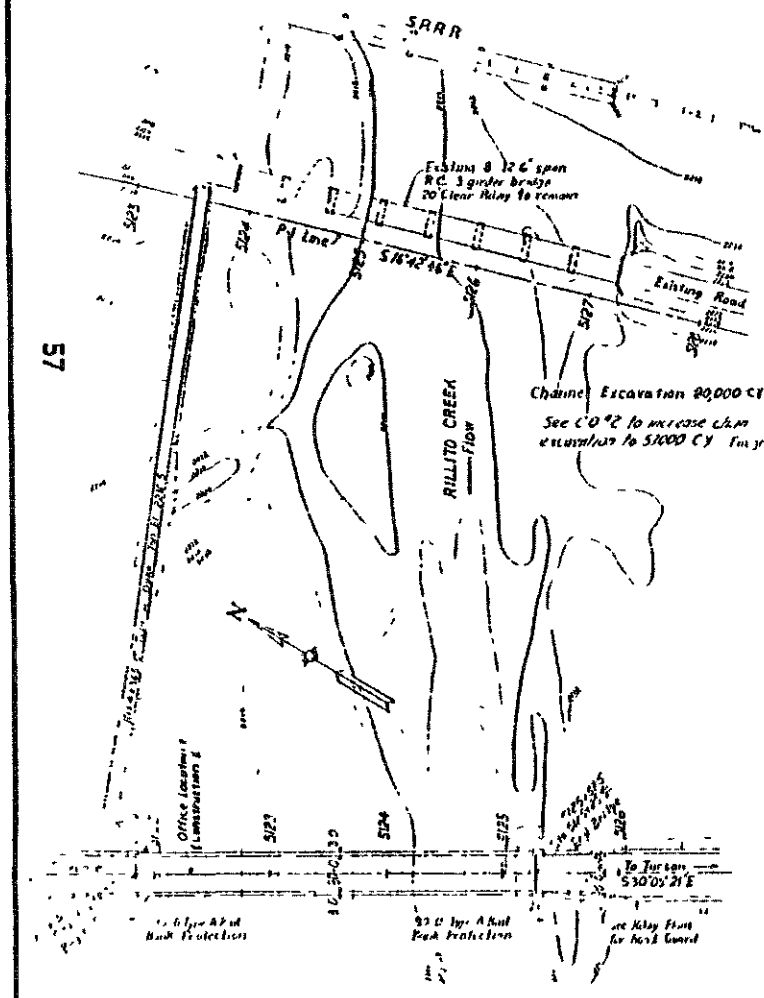
The plans also specified two filter blankets for the riprap.

(ADOT Photogrametry Section)

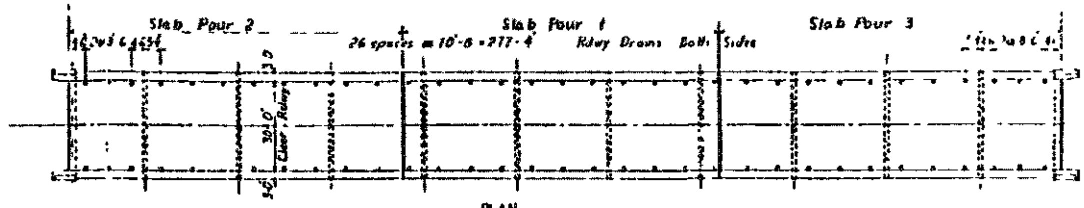


(ADOT Photogrammetry Section)

57

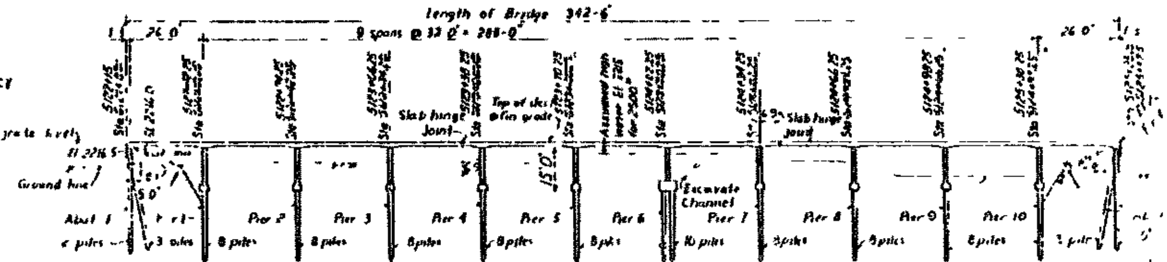


STA 5124+
LOCATION PLAN
Scale 1" = 50'



PLAN
Scale 1" = 20'-0"

See C.O. #1 to move bridge ahead 32'



SECTION ON ℄
Scale 1" = 20'-0"

Piles at abutments shall be driven thru the thoroughly compacted fill. Excavation for abutment concrete shall be made after piles are in place.

Piles may be cast in place Union Metal or approved equal Type TH 16 at piers and Type JH H in abutments or 16 precast concrete at the option of the Contractor. See Std CP-6 Sheet #21. Estimated length of piles: Abut 35', Piers 25'. Pile design load 31 Tons - Piers; 19 " - Abutments.

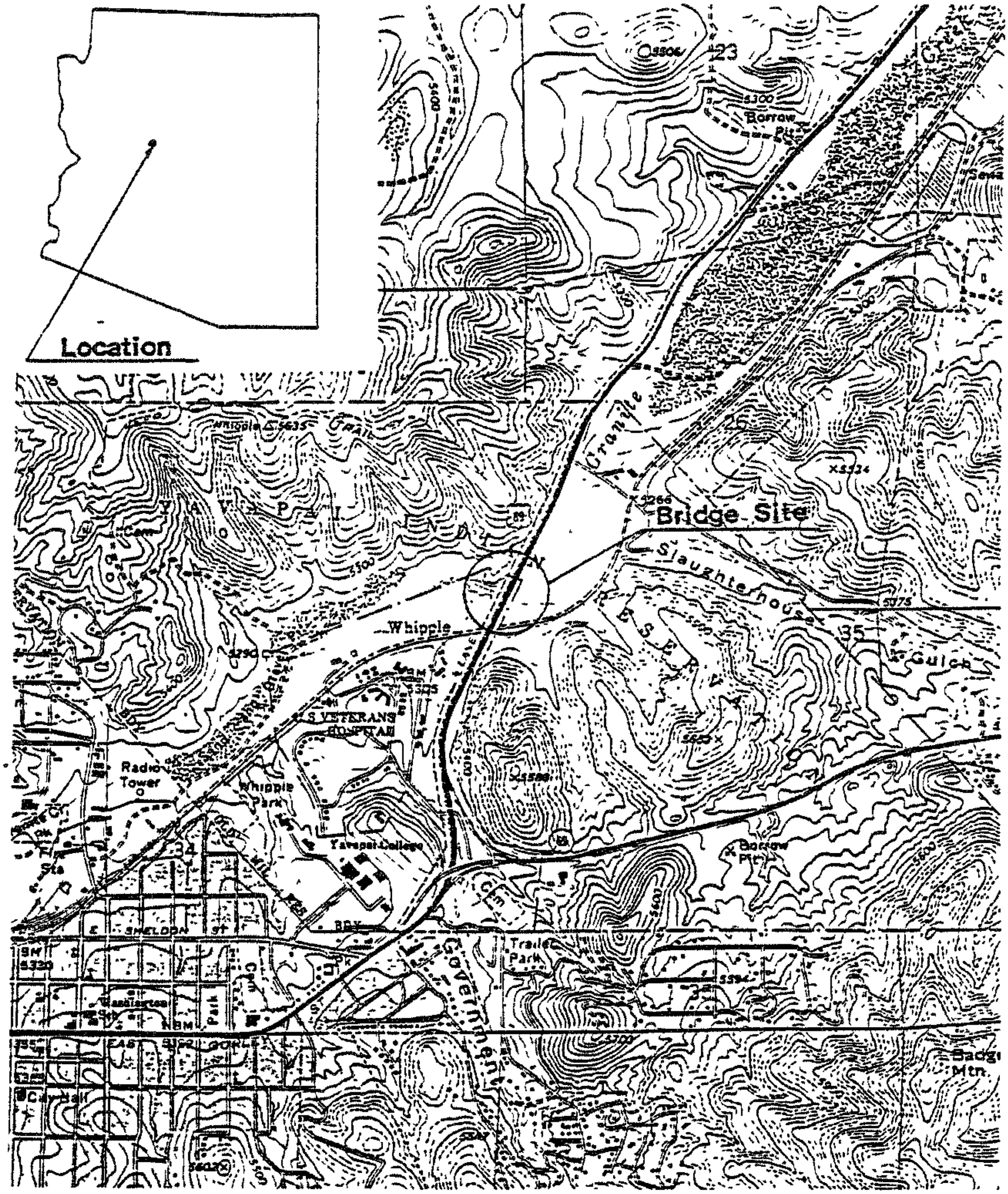
One test pile 35' long of the type to be specified shall be driven in position and used for structure bearing pile at select piers #2, 6 & #10. The number and order length of remaining piles shall be determined by the Engineer after test piles have been driven.

GENERAL NOTES —
Construction: Standard Specifications Arizona Highway Department Edition of 1949
Design: AASHO Specifications 1949 revised to date
Loading Class H 20 S16 1944
Stresses f_s = 20000#/in² f_c = 1200#/in² n = 10 superstructure
f_s = 20000#/in² f_c = 1000#/in² n = 12 substructure
No wearing surface allowance in deck slab design
Structural steel shall conform to A.S.T.M. Specification A 7
Reinforcing steel shall be intermediate grade and shall conform to A.A.S.H.O. Designation M 137 48
All welding shall conform to American Welding Society Specifications for Welded Highway & Railway Bridges
Paint & Painting shall conform to Standard Specifications
Contact surfaces shall not be painted
Shop paint 1 coat paint #1A or #1B
Field paint 1st coat paint #3
2nd coat paint #10
If steel shell piles are used they shall receive [illegible] coats of paint as noted above. See Spec page 184

APPROXIMATE QUANTITIES

ITEM	Std Sheet	Sheet No	Struct Excav CY	Class A	Class D	Steel Reinf	Lbs Struct	Steel Rail lin ft	St Shells No	lin ft	Test Piles No	lin ft
Abutments	[illegible]	20	[illegible]	912	—	540	—	—	10	35	—	—
Piers 2,3,4,5,7-10	[illegible]	20	300	3309	—	[illegible]		—	10	1135	2	70
Pier 6	[illegible]	20	150	4112		[illegible]		—	9	275	1	35
Deck	[illegible]	22	—	—	257	[illegible]	5530	—				—
Hand rail	[illegible]	23	—					[illegible]				—
Total			[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		[illegible]

ARIZONA HIGHWAY DEPARTMENT
BRIDGE DIVISION



CASE HISTORY NO. 5
Granite Creek
Route: US 89

Case History No. 5, US 89, Granite Creek Bridge

Location: 1 mile north of Prescott

Type of crossing:

Two bridges, 211 feet long, each with two piers (3 spans) at a 30° skew to the bridge.

Hydrologic characteristics:

Drainage area = 39 square miles which provides a 50 year highway design flow = 2330 cfs. From as builts: high-water elevation = 5249.0, Q = 6750 cfs. Gauge 5030 is located at the bridge.

Hydraulic characteristics:

The average depth of flow (Q_{50}) = 2.8 feet, with a Froude No. (Q_{50}) = 1.3. The channel makes a 90-degree bend as it passes through the bridge.

General history:

As of 1979, the main channel near pier 1 had degraded seven feet since the original construction. The greatest degradation was noticed during the 1978 inspection. The footings of pier 1 of both structures were exposed and the railbank protection between the structures was undermined. Downstream channel excavation was sited as causing six to eight feet of degradation at the bridge.

Crossing history: 1956

The two bridges are built. The embankments at the base of the bridge abutments are protected with riprap. On the north abutment, 260 feet of riprap was installed and on the south abutment 300 feet of a railbank protection was installed. The size gradation of dimensions of the riprap protection were not shown. The protection was placed on a 1.5:1 fill slope with the railbank protection having a three-foot vertical toedown, the riprap appears to have been installed at a 2:1 slope. The bottom width of the opening under the bridges is about 100 feet. From the plans, the southern abutment is located on the outside bank of the channel bend.

1976:

First bridge inspection report: Only the railbank on the south abutment is mentioned and it is reported as in satisfactory condition. The riprap that was on the north abutment is not mentioned. In the channel, one foot of scour was reported at midchannel.

1978:

April, bridge inspection report: The railbank protection on the south bank was reported in good condition. The channel bed had degraded by five feet at the footing of the south pier. Extensive scouring had exposed the footings of the pier columns. The inspection cautioned that possible trouble could occur in the near future.

1980:

The bank protection at the base of the south abutment was rebuilt as rock and rail, standard C-17.02. A grade control sill was constructed at the downstream side of the bridges. The sill was set to raise the channel invert through the bridges by eight feet, allowing four feet of fill above the top of the pier footings on the south bank. A stilling basin was constructed below the sill to dissipate energy from the sudden drop in channel profile. The sill was constructed of rock and rail with a concrete cap. There are a series of drops into the stilling basin, a six-foot drop with a +17-foot-long riprap apron, then two more drops and aprons, each with a 2.5-foot drop and 6.5-foot aprons made of wire-tied riprap. Rock was specified as sound and durable, of rounded or angular shape and nominal diameter of eight inches minimum and 21 inches maximum, flat or needle shapes were not acceptable. Rock for the drop structures was specified to include at least 50% of eight inch to 12 inch and 5% at 18 inch to 21 inch. Other rock used was to meet standard RB-2 and include filter fabric.

1982:

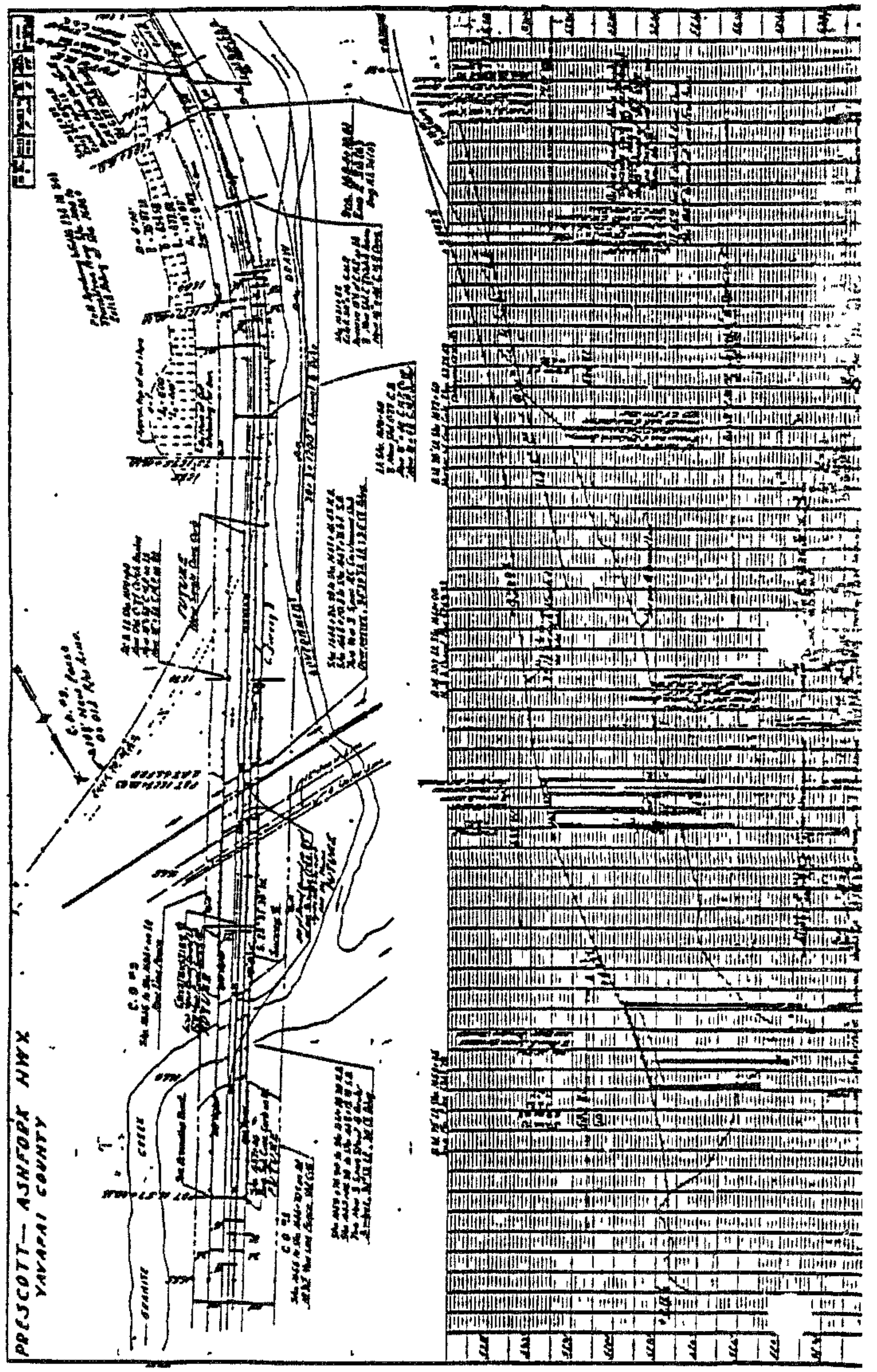
Movement and loss of rock in the stilling basin occurred.

1982:

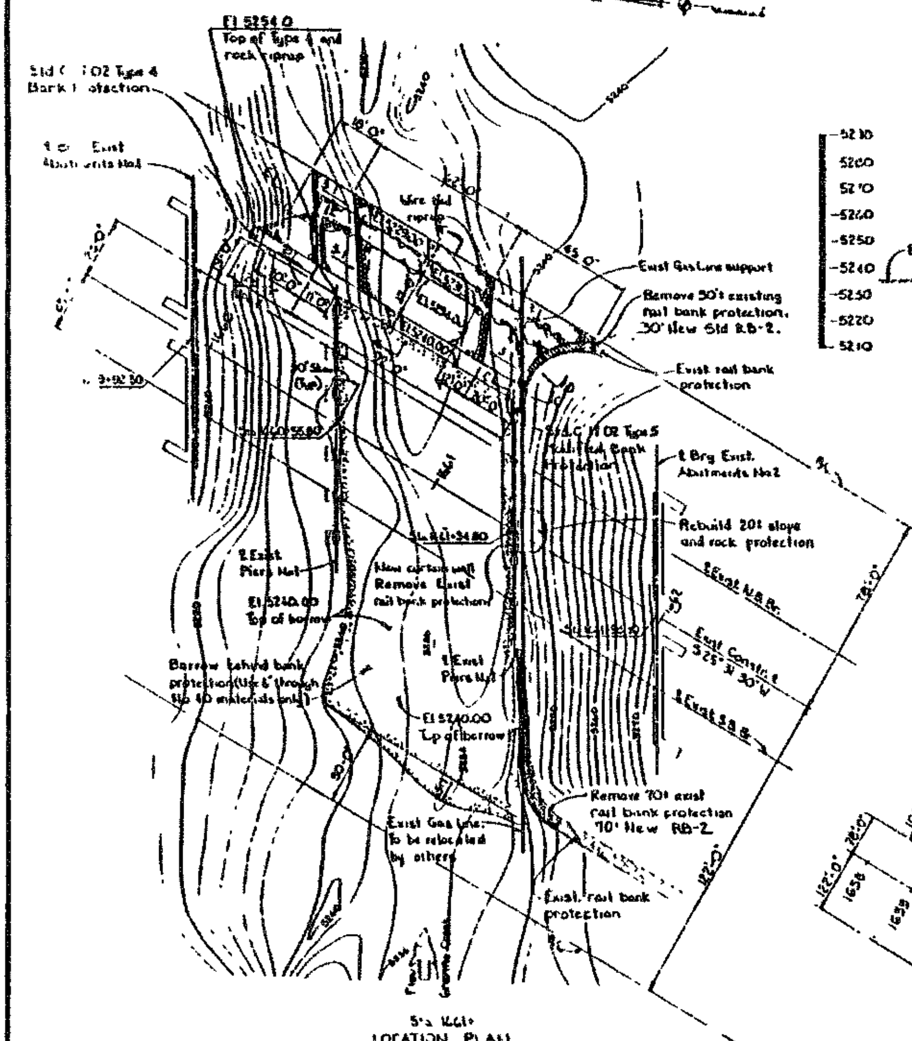
Grouted spillway was completed. A large grouted riprap stilling basin is completed where the drop structure of 1980 was built 45 feet long by 130 feet wide.

1985:

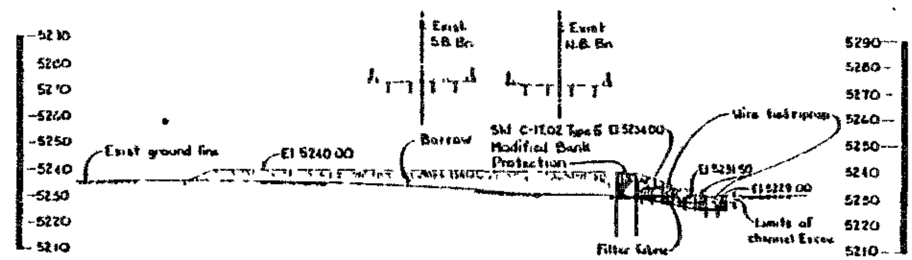
Bridge inspection report: The downstream drop structure and stilling basins are working well. The channel has remained stable.


PRESCOTT— ASHFORK HWY
YAVAPAI COUNTY

PRESCOTT-ASHFORK HWY
YAVAPAI COUNTY



Sta 1661+
LOCATION PLAN
Contour Interval = 2'-0"
Scale: 1"=20'-0"



SECTION ALONG CENTER OF STREAM
Scale: 1"=20' 0"

APPROXIMATE QUANTITIES		
Channel Excavation	185	Cu Yds
Borrow (Behind Modified Type 5 Bank Protection)	1805	Cu Yds
Structural Excavation	20	Cu Yds
Exist Rail Bank Prot - Remove	120	Lin Ft
New Rail Bank Prot, Std RB-2	100	Lin Ft
Std C 17.02-Type 5 Modified	92	Lin Ft
Std C 17.02 Type 4	28	Lin Ft
Wire-tied Riprap	355	Cu Yds
Class "S" Concrete	24.1	Cu Yds
Reinforcing Steel	1560	Lbs.
Place Dowels	26	Each

GENERAL NOTES:

Construction-Std Specs Arizona Highway Department Edition of 1969, revised to date

Stresses- f'c=3000 psi-Class "S" Concrete.

fs =20000 psi-Reinforcing Steel

Use Class "S" Concrete for curtain wall and footing and for bank protection cap.

Reinforcing Steel shall conform to ASTM Spec. A615, Grade 40.

All reinforcing steel dimensions shall be to center of bars unless noted otherwise

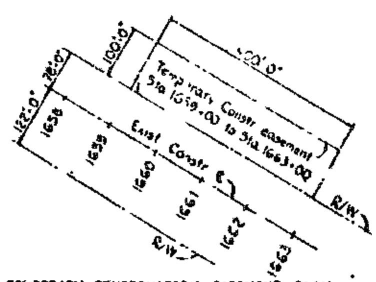
All reinforcing steel shall have 2" clear cover unless noted otherwise.

All stations and elevations shown indicate existing data on "As-built" plans and do not necessarily correspond to structure conditions now existing

Excavation materials shall be disposed of at the site under the direction of the Engineer.

Contractor to verify all dimensions in the field prior to the fabrication of materials.

Existing rails, rock, and wires shall be reused or disposed of as directed by the Engineer.



TEMPORARY CONSTRUCTION EASEMENT PLAN
Scale - 1"=100'-0"

SITE NO 5
JOINT FHWA/ADOT
BRIDGE INSPECTION
JUNE 1979

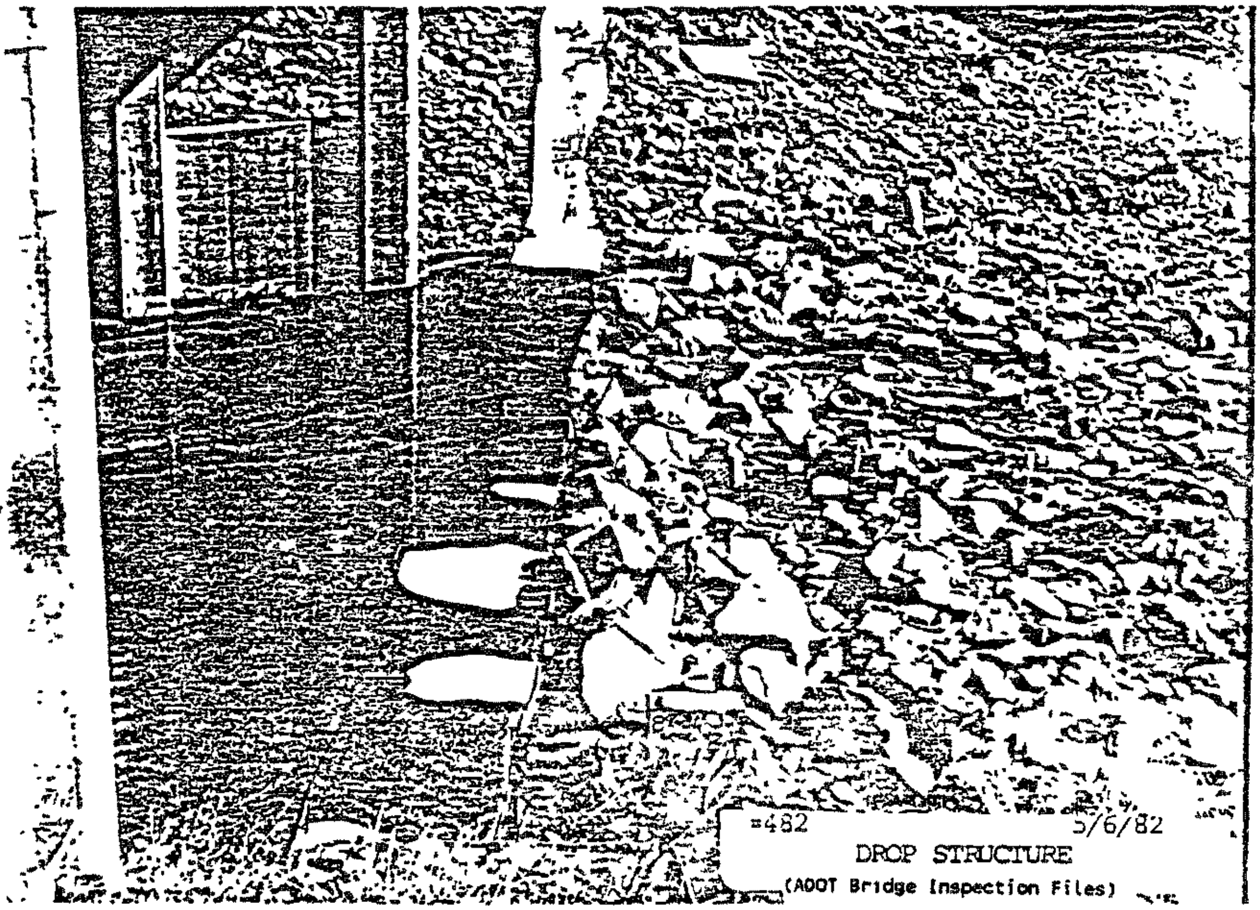
DO NOT DESTROY

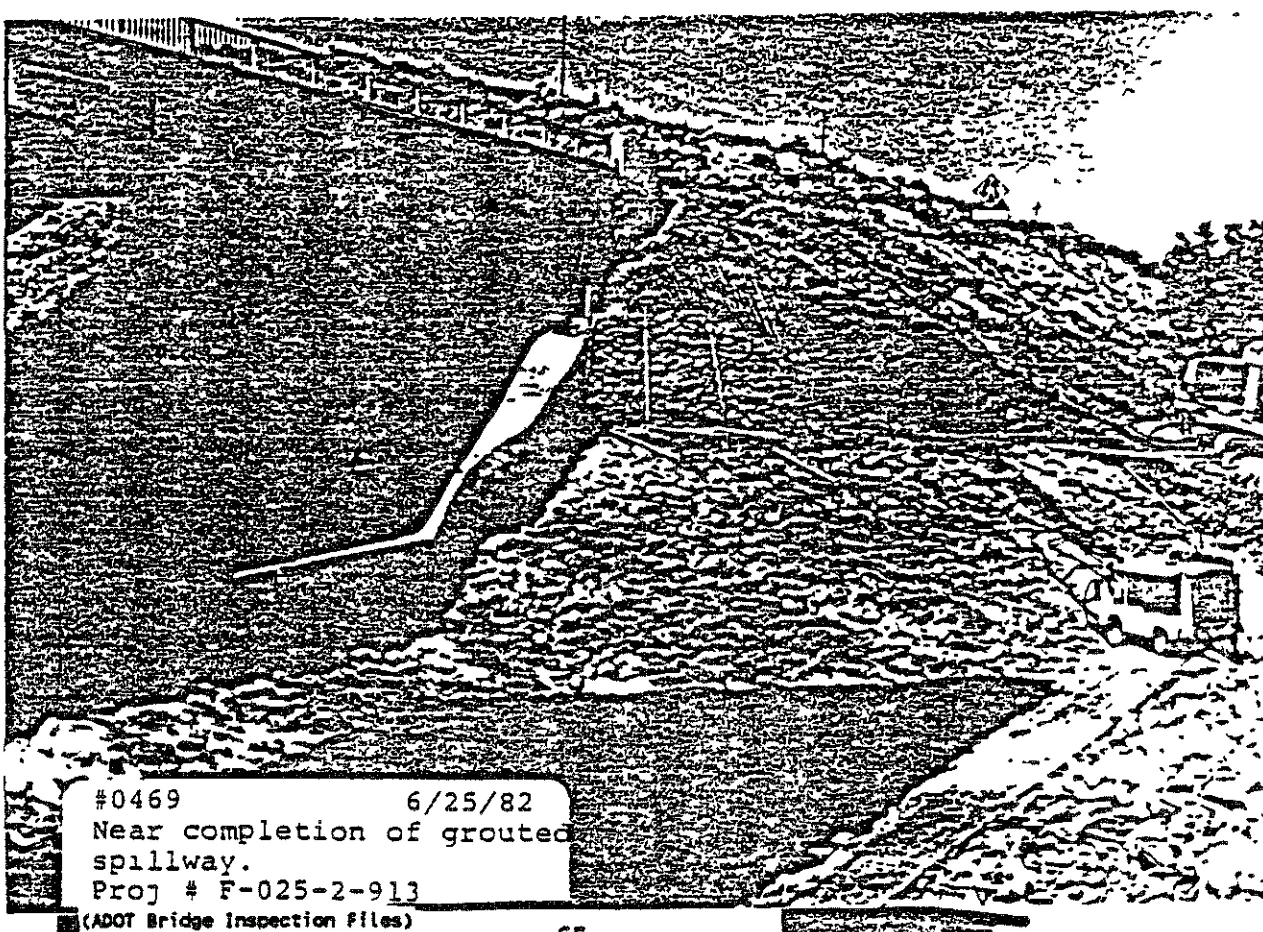
For Sections A-A, C-C, D-D & E-E See sheet 4 of 5

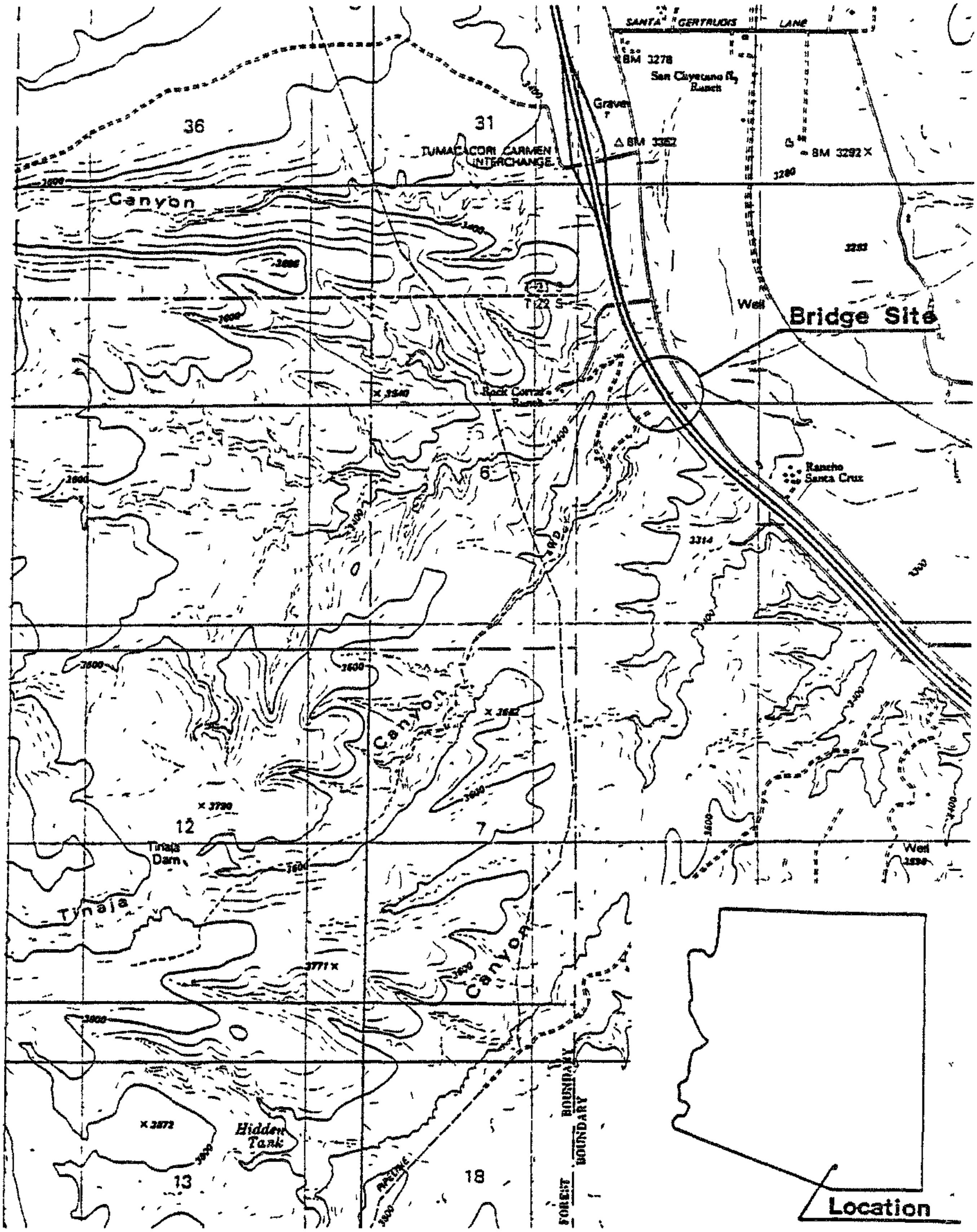
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
STRUCTURES SECTION
STA 1661+
GRANITE CREEK BRIDGES
SCOUR PROTECTION DETAILS
F-025 2 911

63

(ADOT Photogrammetry Section)


#482 5/6/82
DROP STRUCTURE
(ADOT Bridge Inspection Files)


#0469 6/25/82
Near completion of grouted spillway.
Proj # F-025-2-913
(ADOT Bridge Inspection Files)



CASE HISTORY NO. 6

Tinaja Wash

Route: I-19

Case History No. 6, I-19, Tinaja Wash Bridges

Location: Approximately 45 miles south of Tucson

Type of Crossing:

Three bridges: northbound, southbound and east frontage road bridge. Each bridge is approximately 129 feet long, with three piers (4 spans). These structures are four-span, reinforced concrete, continuous slab bridges with approximately a 35° skew.

Hydrologic characteristics:

Drainage area = eight-square miles which provides a Q_{50} = 2820 cfs and a Q_{100} = 2990 cfs. These flow values were determined from an ADOT drainage study in May of 1975.

Hydraulic characteristics:

Designing for the Q_{50} flow, the average velocity was approximated at 10.5 ft/sec. The depth of flow under these conditions was determined to be approximately 3.2 feet.

Crossing history:

When these three structures were built in 1975, the contract included bank protection. This bank protection consisted of dumped riprap placed along both banks beginning upstream of the southbound bridge to the downstream side of the east frontage road bridge.

1983:

In October of the year, a major storm event was experienced. From maintenance records of the high water mark and use of the slope area method, it was determined that a flow of approximately 3000 cfs was developed. Upon inspection of the structures after the flood, the following damages were recorded:

1. Lateral erosion to the south bank, between the west frontage road and southbound bridges. The lateral scour caused severe damage to the existing dumped riprap.

2. The north bank of the southbound bridge experienced minor riprap damage.
3. The south end of the southbound and the north end of the northbound bridges also experienced riprap damage.

1984:

In late February and early March of 1984, ADOT conducted a backhoe investigation at several sites along I-19 relative to the October 1983 flood. At Tinaja, trenches were excavated to determine the extent of damage to existing riprap installations. Both trenches were dug at the banks, west of the southbound structure.

The north bank exposed riprap protection immediately below the bank surface. The excavation also revealed that most of the original section is intact. During the excavation of the south bank trench, no evidence of pre-existing riprap was found. Following the backhoe investigation, ADOT made the following recommendations:

1. Because the north bank riprap damage was minor, it was recommended the riprap be regraded and grouted 20 feet upstream and downstream of the southbound bridge. Estimated 90% of original riprap section was still intact.
2. The south bank riprap damage occurred from the centerline of the southbound bridge to its upstream terminus. Regrade and grout bank 20 feet downstream of southbound bridge to railbank. Estimated 50% of original riprap section was still intact.

Riprap design: 1984

The method used by ADOT was one which was developed by Ray Jordan, the hydraulics-group section head at the time. The developed equation used:

$$D_{50} = 0.0006086 \frac{V^3}{y^{\frac{1}{2}}} (K_{sf}) (K_{sg}) (K_{ss}) (K_b)$$

This equation was derived from equations in various publications. Using this design method, D_{50} was determined to be 12 inches. The following gradation was also recommended:

Percent Passing	Rock Size
100	18"
40-60	12"
20-40	9"
0-5	3"

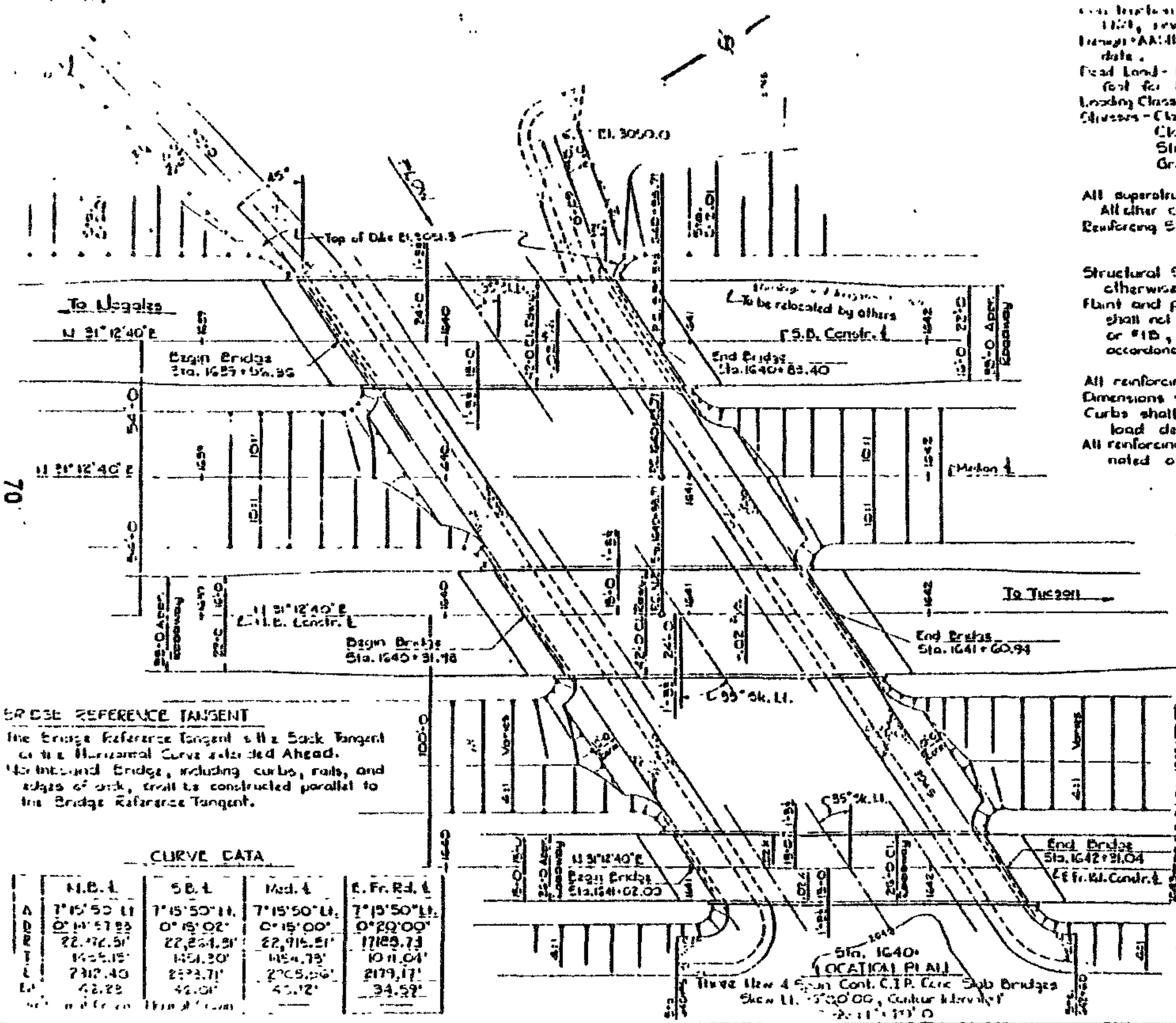
GENERAL NOTES:

Construction: Std. Specs. Arizona Highway [illegible] Edition 1969, revised to date.
Design: AASHO Std. Specs. for Highway Bridges 1969, revised to date.
Dead Load: Dead load includes allowance of [illegible] per sq. foot for future wearing surface.
Loading Class: HS20-44 and/or Interstate Alternate Loading.
Stresses: Class A concrete f'c = 1000 psi
Class B concrete f'c = 1200 psi
Structural Steel fs = 20,000 psi
Gr. 40 Reinf. Steel fs = 20,000 psi

All superstructure concrete except curbs and parapets shall be Class [illegible]. All other concrete including curbs and parapets shall be Class A.
Reinforcing Steel shall conform to ASTM Spec. A615, Grade 40.

Structural Steel shall conform to ASTM Spec. A36, unless noted otherwise.
Paint and painting shall conform to Std. Specs. Contact surfaces shall not be painted. Structural Steel-Shop-1 coat paint #1A or #1B, #9 and #10. Exposed area of piles shall be painted in accordance with Std. Spec. 512-3.08.

All reinforcing steel shall have 2" clear cover unless noted otherwise.
Dimensions shall not be scaled from drawings.
Curbs shall be constructed after spans have taken slab dead load deflection.
All reinforcing steel dimensions shall be to center of bars unless noted otherwise.



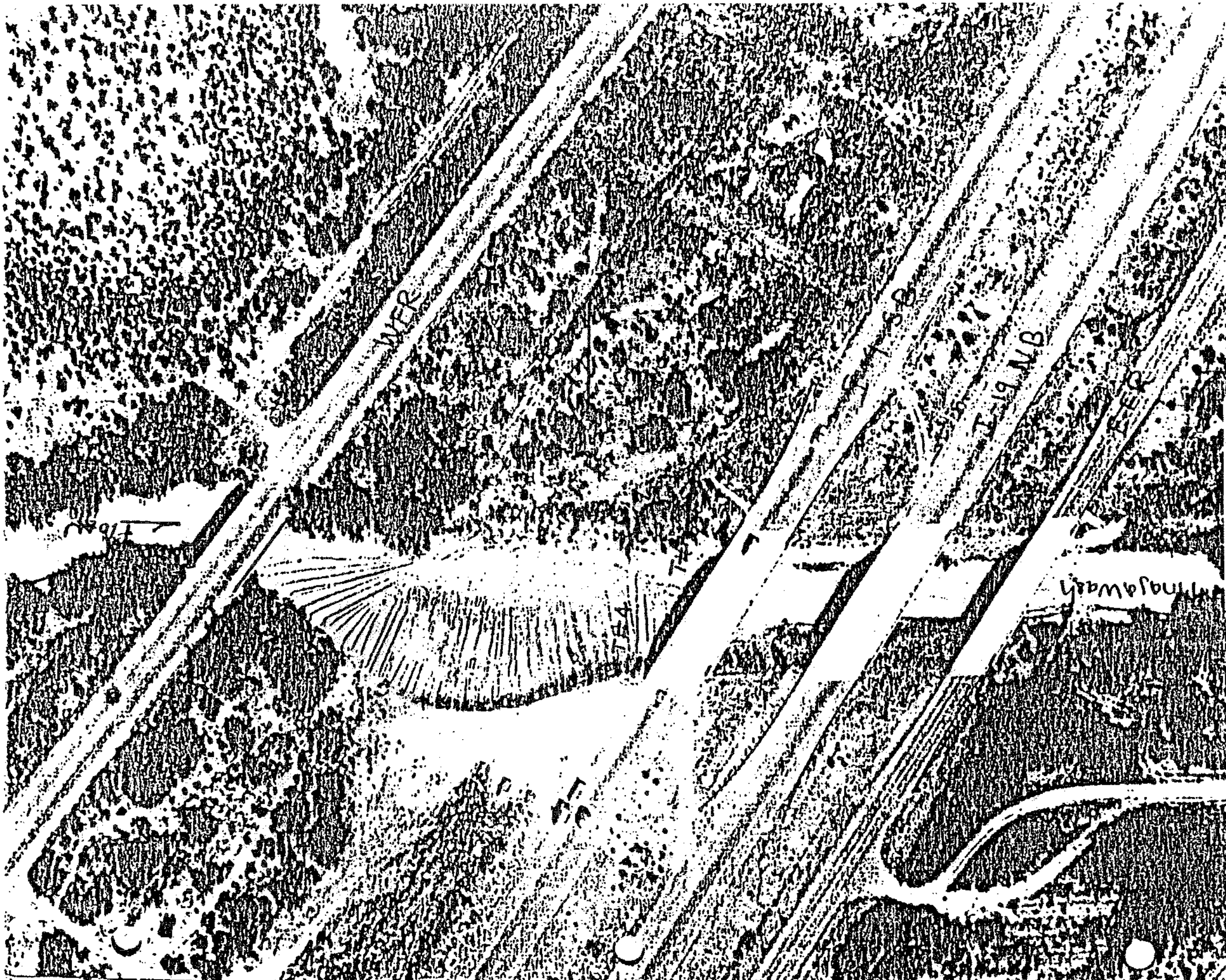
BRIDGE REFERENCE TANGENT

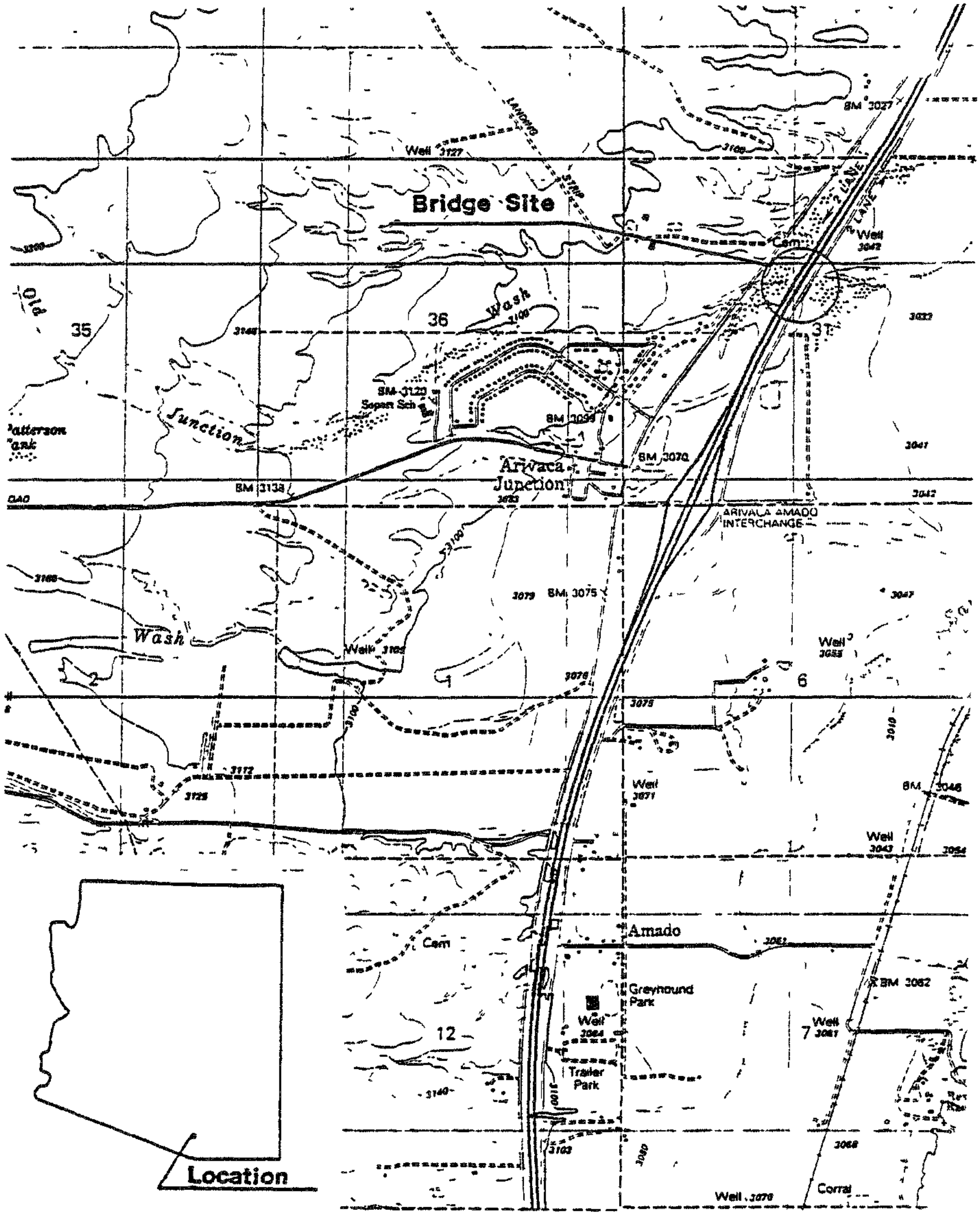
The Bridge Reference Tangent is the Back Tangent of the Horizontal Curve extended Ahead.
Northbound Bridge, including curbs, rails, and edges of deck, shall be constructed parallel to the Bridge Reference Tangent.

CURVE DATA

	N.B. ℄	S.B. ℄	Med. ℄	E. Fr. Rd. ℄
Δ	7°15'50" Lt.	7°15'50" Lt.	7°15'50" Lt.	7°15'50" Lt.
D	0°14'57.85"	0°15'02"	0°15'00"	0°20'00"
R	22,[illegible]	22,854.51'	22,916.51'	17189.73
T	1455.15'	1451.30'	1454.78'	1091.04'
L	2912.40	2903.71'	2905.56'	2179.17'
E	46.25	46.01'	46.12'	34.59'

ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
STRUCTURES SECTION
Sta. 1640
TINAJA WASH BRIDGES
LOCATION PLAN


WFR
I-19 NB
ADOT Photogrammetry Section



CASE HISTORY NO. 7

Old Junction Wash

Route: I—19

Case History No. 7, I-19, Old Junction Wash

Location: Approximately 32 miles south of Tucson

Type of crossing:

Three bridges: northbound, southbound and east frontage road. Each bridge is 129 feet long, with three piers (4 spans) at a 35° skew to the flow. The structures are four-span, continuous cast-in-place concrete slab bridges.

Hydrologic characteristics:

Drainage area = six square miles, which provides a Q_{50} = 2530 cfs and a Q_{100} = 2700 cfs. The flow from this wash is channelized upstream of U.S. Highway 89 with bank protection on both the north and south banks.

Hydraulic characteristics:

The average depth of flow is 3.4 feet with an average velocity of 8.75 ft/sec. With the channelization of the wash under U.S. 89, the wash does widen under the I-19 structures.

Crossing history: 1983 - Q = 3800 cfs (estimated by maintenance crews from high water marks)

The west frontage road, old U.S. 89, sustained no damage to bridge or railbank of which consisted of 179 feet on the north and 336 feet on the south banks respectively. Most of the riprap losses occurred at the transitions from dumped riprap to wire-tied riprap at the north abutment of the southbound bridge. Two feet of degradation was recorded in the stream channel but an estimated 80% of the riprap is still intact.

Recommendations:

1. New grouted riprap upstream and downstream on north bank of southbound Interstate, placed in front of and against existing wire-tied.
2. Reconstruct 125 feet of dumped and grouted riprap along north bank for 25 feet upstream and 60 feet downstream.
3. Reconstruct 250 feet dumped and grouted riprap along south bank of northbound Interstate, placed 25 feet upstream to about 40 feet downstream of east frontage road.

Riprap design: 1975

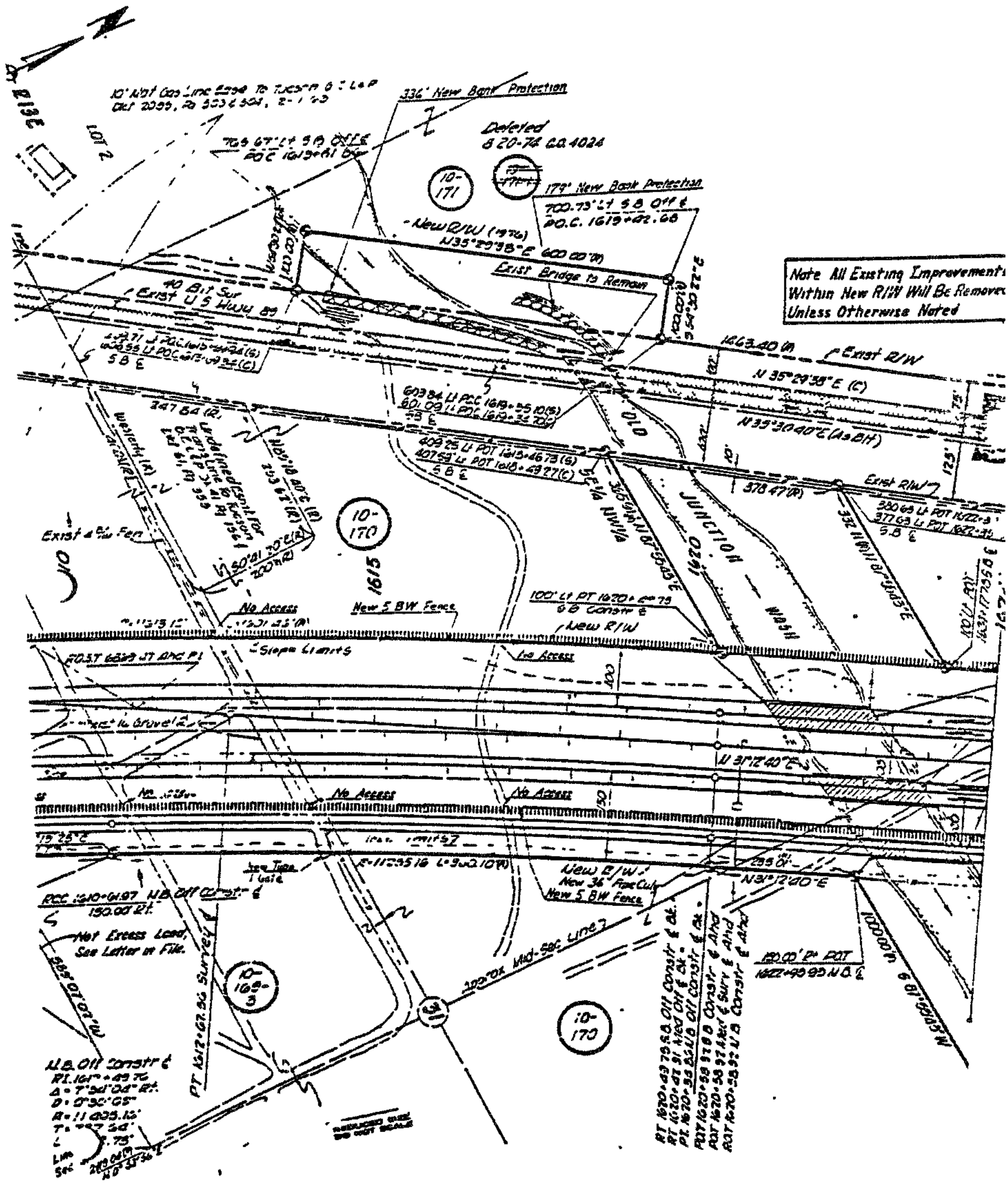
The method used by ADOT was one which was developed by Ray Jordan, the hydraulics-group section head at the time. The developed equation:

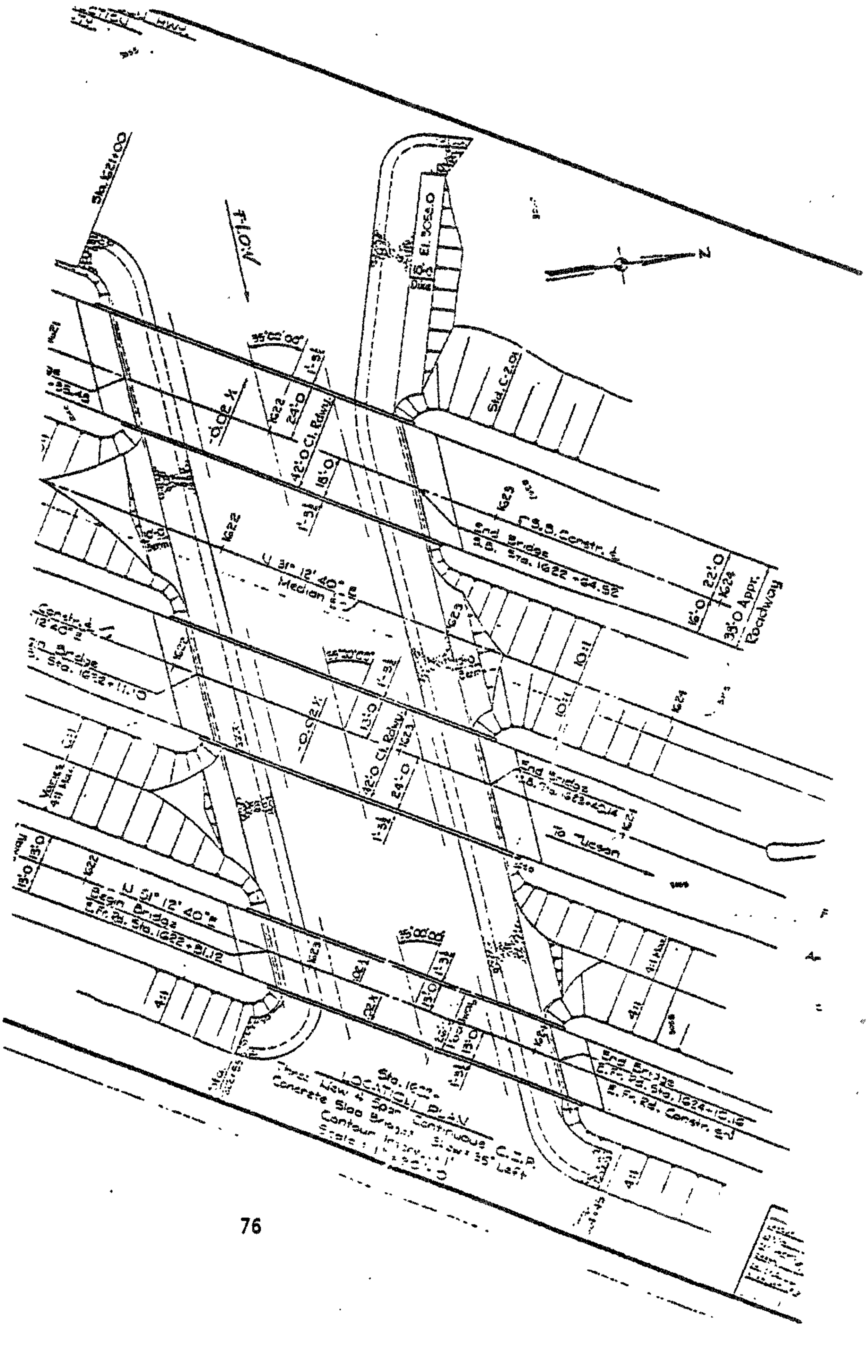
$$D_{50} = 0.0006086 \frac{V^3}{y^{\frac{1}{2}}} (K_{sf}) (K_{sq}) (K_{ss}) (K_6)$$

This equation was derived from equations in various publications. The major references for this method are as follows:

1. Lawsen, E.M.; "An Analysis of Relief Bridge Scour"; Proceedings of ASCE, 1963; pg 96.
2. F.H.W.A., "Highways in the River Environment"; 1975; p VIA-12.
3. F.H.W.A., "Hydraulic Engineering Circular No. 11", 1967; p 11-14.
4. F.H.W.A., "Hydraulic Engineering Circular No. 15", 1975; p 15-16.

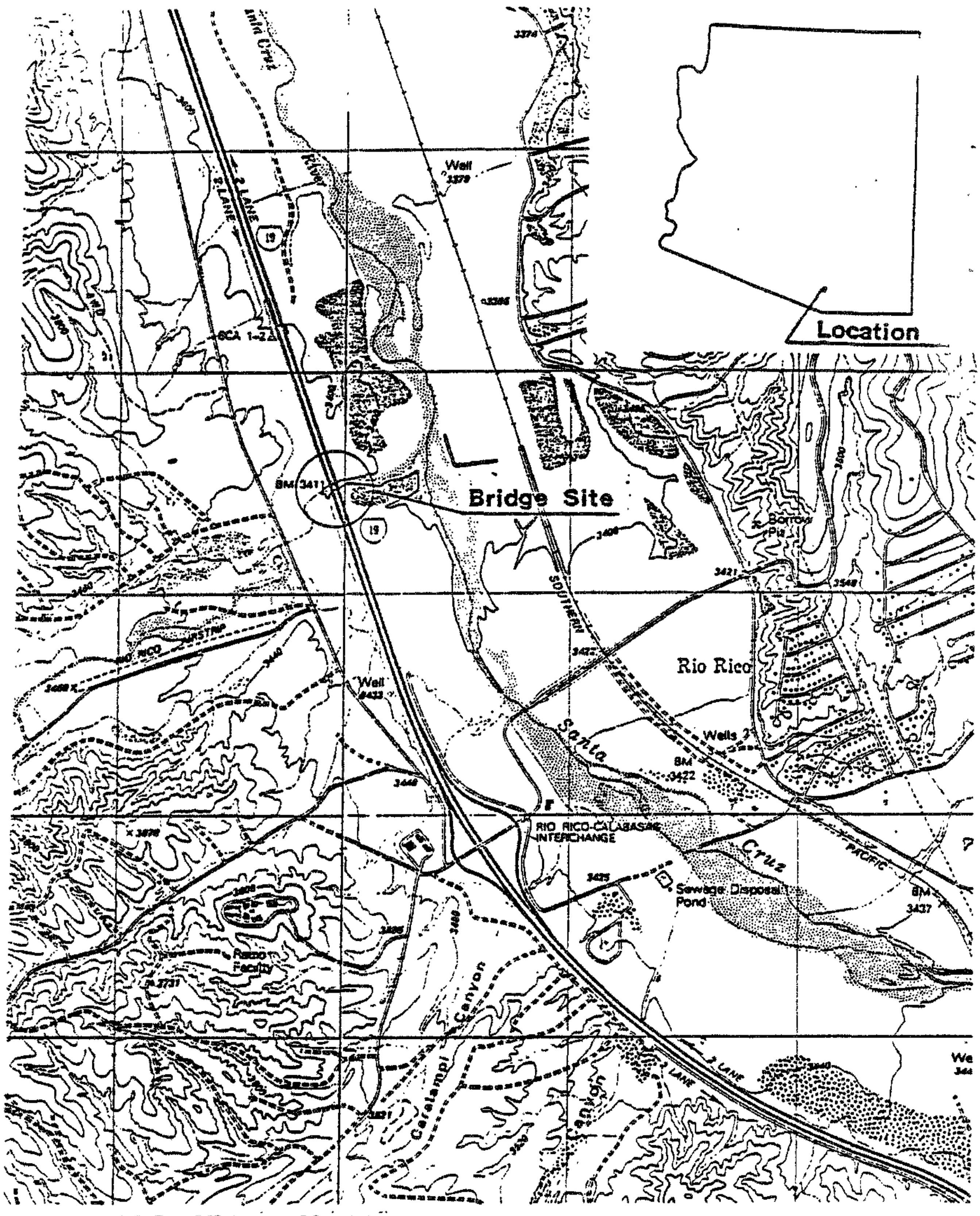
Using the above derived equation, ADOT determined that a $D_{50} = 12"$ would be adequate.


336' New Bank Protection
Deleted
8 20-74 C.O. 4024
179' New Bank Protection
10-
171
New R/W (1976)
N35°29'38"E 600.00'(M)
Exist Bridge to Remain
Note All Existing Improvements
Within New R/W Will Be Removed
Unless Otherwise Noted
40 Bit Sur
Exist U S Hwy 89
Exist R/W
N 35°29'38" E (C)
N 35°30'40"E (As BH)
Exist R/W
10-
170
1615
1620
Exist 4 Str Fen
No Access
New 5 BW Fence
New R/W
No Access
No Access
No Access
New R/W
New 36" Pipe Culv
New 5 BW Fence
Mid-Sec Line
Not Excess Land,
See Letter in File.
10-
169-
3
10-
170
N.B. Off Constr ₵
PT 1612+67.56 Survey ₵



Sta. 1621+00
Std. C-2.01
E.S.B. Constr. ℄
33'-0 Appr. Roadway
42'-0 Cl. Rdwy.
24'-0
To Tucson

(ADOT Bridge Inspection Files)



CASE HISTORY NO. 8

Agua Fria Canyon
Route: I-19

Case History No. 8, I-19, Agua Fria Canyon

Location: Approximately 12 miles north of Nogales

Type of crossing:

Two bridges, northbound and southbound. Each bridge is 92 feet long, with three piers (4 spans) at a 0° skew. Both bridges are of reinforced concrete slab construction.

Hydrologic characteristics:

Drainage area = 43 square miles which provides a design flow of 10,600 cfs. This design flow is the storm of record and was obtained from slope area calculations by the U.S.G.S. ADOT estimated Q = 10,200 cfs from high water marks on the bridge.

Hydraulic characteristics:

The depth of flow was 6.2 feet during the flow of record with an average velocity = 12 ft/sec. At the location of these bridges, the river could be characterized as a meandering incised alluvial channel. The most recent improvements to the channel's stability include spur dikes and a concrete channel lining between and under the structures.

Crossing history: 1977

The October 9th flood was of such magnitude that the southbound structure had to be closed because of the loss of the southwest corner of the southbound approach due to scour.

The frontage road bridge, Old U.S. 89, was located 700 feet upstream of I-19 and washed out on October 9th. The bridge abutments and upstream railbank remained intact. The high water elevation was estimated to have been about equal to the top deck elevation of the frontage road bridge. It was suspected that the bridge failed because the spread footings supporting the piers were undercut by scour. A 100-foot overflow section, adjacent to the south abutment, was not of sufficient capacity to prevent the loss of the bridge.

During the flood, approximately one foot of water overflowed the Interstate roadway at the north abutment of the southbound bridge. This overtopping washed out a section of roadway embankment and undercut the approach slab to the southbound bridge. The north channel bank approaching the southbound bridge scoured laterally about 20 to 30 feet.

Immediately upstream of the northbound bridge, the high water elevation was about even with the northbound top deck elevation, however, no flow overtopped the northbound roadway. Minor scour occurred behind the downstream wingwalk of both the northbound and southbound bridges. Scour also occurred behind the north inlet wingwalk of the northbound structure.

Using the high water marks upstream of the southbound structure, assuming no velocity of approach and the existing groundline, a peak discharge of about 11,000 to 12,000 cfs is estimated to have occurred. The discharge may be affected by storage at Pena Blanca Lake which is located about seven miles upstream.

1983:
The October 1983 flood caused less damage to the structures. Along the northwest bank, upstream of the bridges, 125 feet of railbank was scoured. Most of the rails were present after the flow but slightly bent. A small amount of fill was gone behind the northwest wingwall of the southbound bridge. About 30 feet of the spur dike suffered scour damage. The flood waters had penetrated the rock and filter fabric and removed a good deal of the compacted fill.

On April 1st, a backhoe investigation was performed to assess subsurface damage to the bank protection. Two trenches were dug, one at the nose of the washed out portion of the south spur dike, and the second at the centerline of the 42 inch CMP's going through the dike. At the first location, riprap and filter fabric were found 20 feet into the channel and just below the surface. The second excavation found the top of the riprap toe trench seven feet below the channel. In 1986, the nose of the spur dike was grouted as recommended by this investigation.

Riprap design:

For the protection of the abutments, the riprap size was determined through the use of the CSU method. With the previously mentioned hydraulic parameters and the following input:

Angle of repose = 41.5°
Safety factor = 1.25
Side-slope = 2:1
G = 2.48
D_{50} = 2 feet

From the calculations, the following gradation was specified:

Percent Passing	Rock Size
100	38"
40-60	24"
20-40	18"
0-5	6"

Using E.M. Laursen's, "An Analysis of Relief Bridge Scour", ADOT designed the channel riprap blanket to reduce scour at the abutments to five feet. Using this procedure, the riprap size determined was too large to be economically feasible. It was recommended that a reinforced concrete lined channel be used. In 1982, the channel was lined between and under both structures.

Also, at this time, a design was performed using HEC-15. By this method, D_{50} = 10 inches for the bed protection and D_{50} = 12 inches for the channel sides. The decision was made to use the conservative D_{50} = 24 inches previously calculated for the abutments.

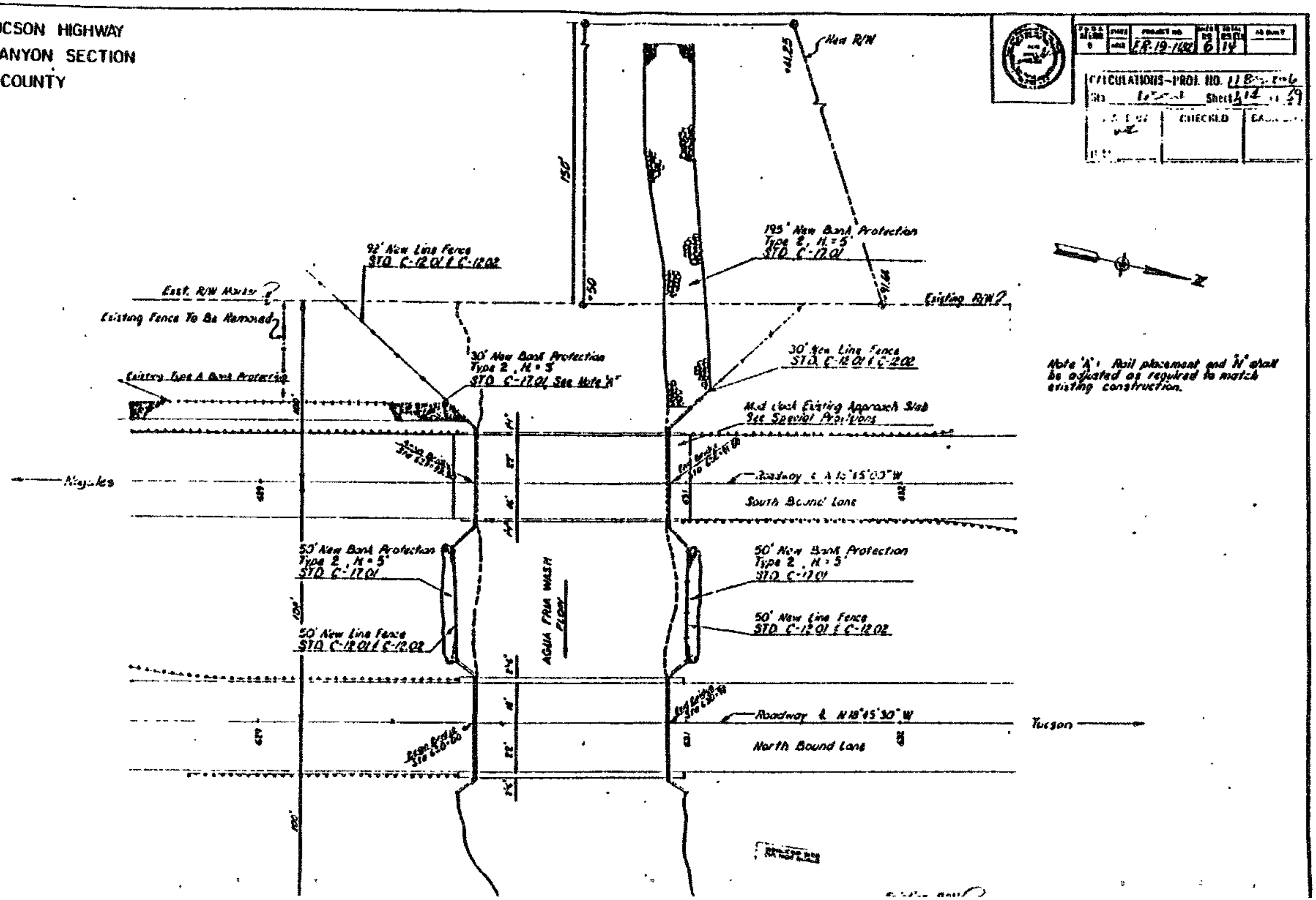
1978:

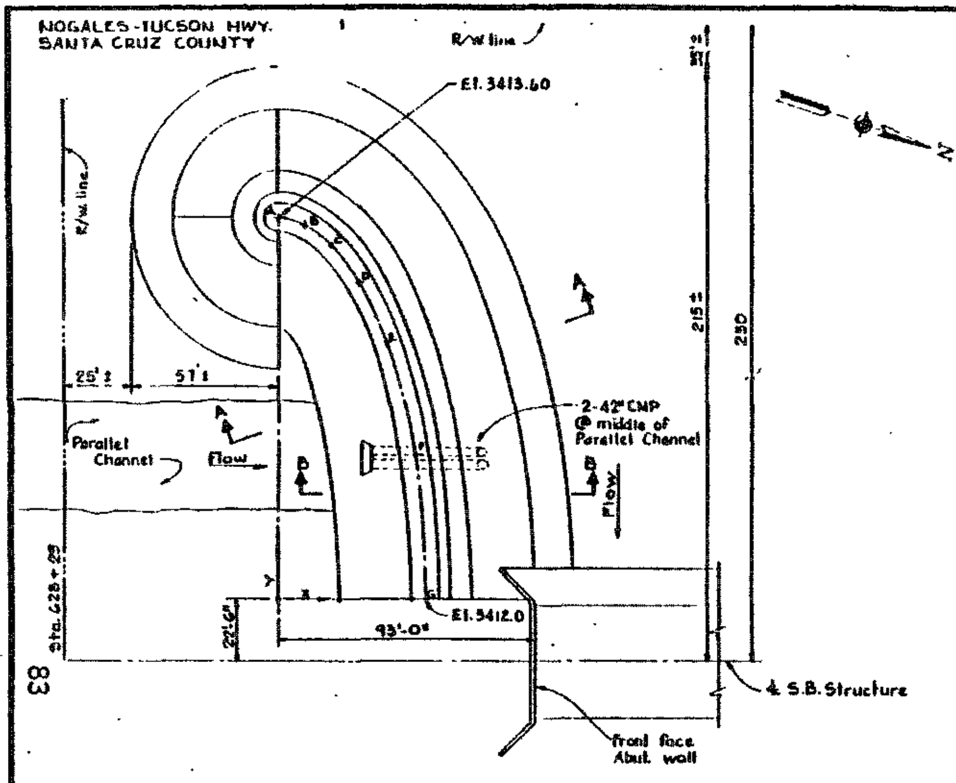
In January, a riprap dissipator pool was designed using HEC-14. The following parameters were used by ADOT engineers:

Q = 754 cfs
Tw = 6.2 feet
V_n = 7.4 ft/sec
Y_e = 5.0 feet
A_b = 50 feet^2

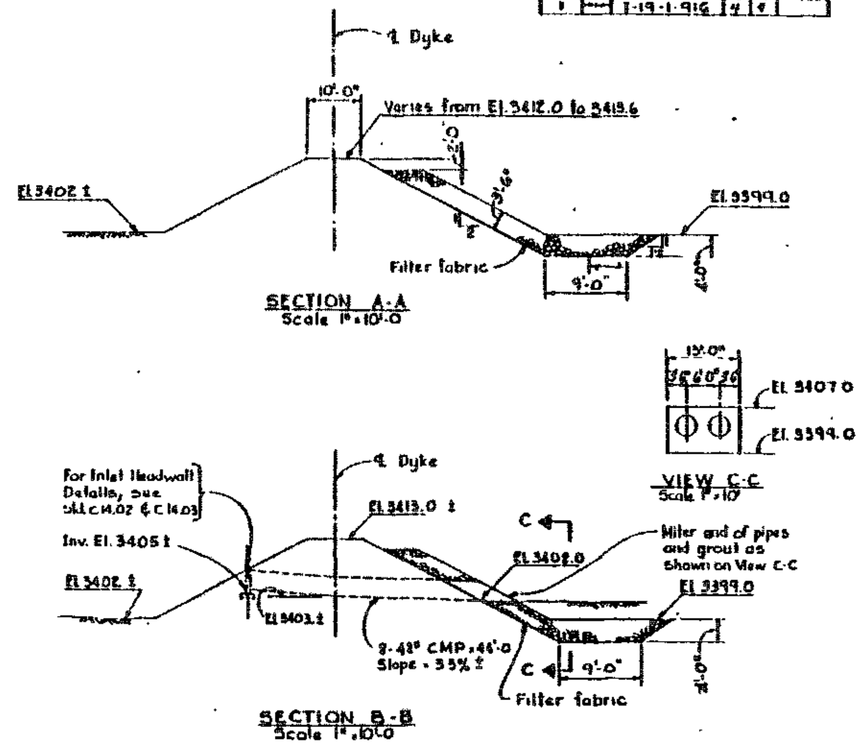
The length of the dissipator pool was determined to be 44.5 feet with an apron length of 17.5 feet.

- TUCSON HIGHWAY
A CANYON SECTION
UZ COUNTY





SPUR DYKE PLAN
Scale 1" = 20'-0"



Rip Rap Rock Gradation	
% Passing	Size
100	30"
40-60	24"
20-40	18"
5-20	12"
0-5	6"

Rock shall be durable & angular with minimum Sp gravity = 2.48

SPUR DYKE LOCATION DATA							
POINT	A	B	C	D	E	F	G
X-Axis	0	10'	20'	30'	40'	50'	54'
Y-Axis	135.0'	132.67'	125.4'	112.25'	90.69'	50.99'	0

APPROXIMATE QUANTITIES (DYKE)		
Rock Rip Rap	1820	Cu. Yd.
† Compacted Dyke fill *	2180	Cu. Yd
42"Φ C.M.P.	88	Ft.
⊕ Class A Concrete (Headwall)	4.0	Cu Yd
Reinforcing Steel (Headwall)	147	Lbs.
† Special Backfill (CMPs)	150	Cu.Yd.

*Compact in accordance with Specs.
†For Informational purposes only.
⊕Included in Item 5020052.

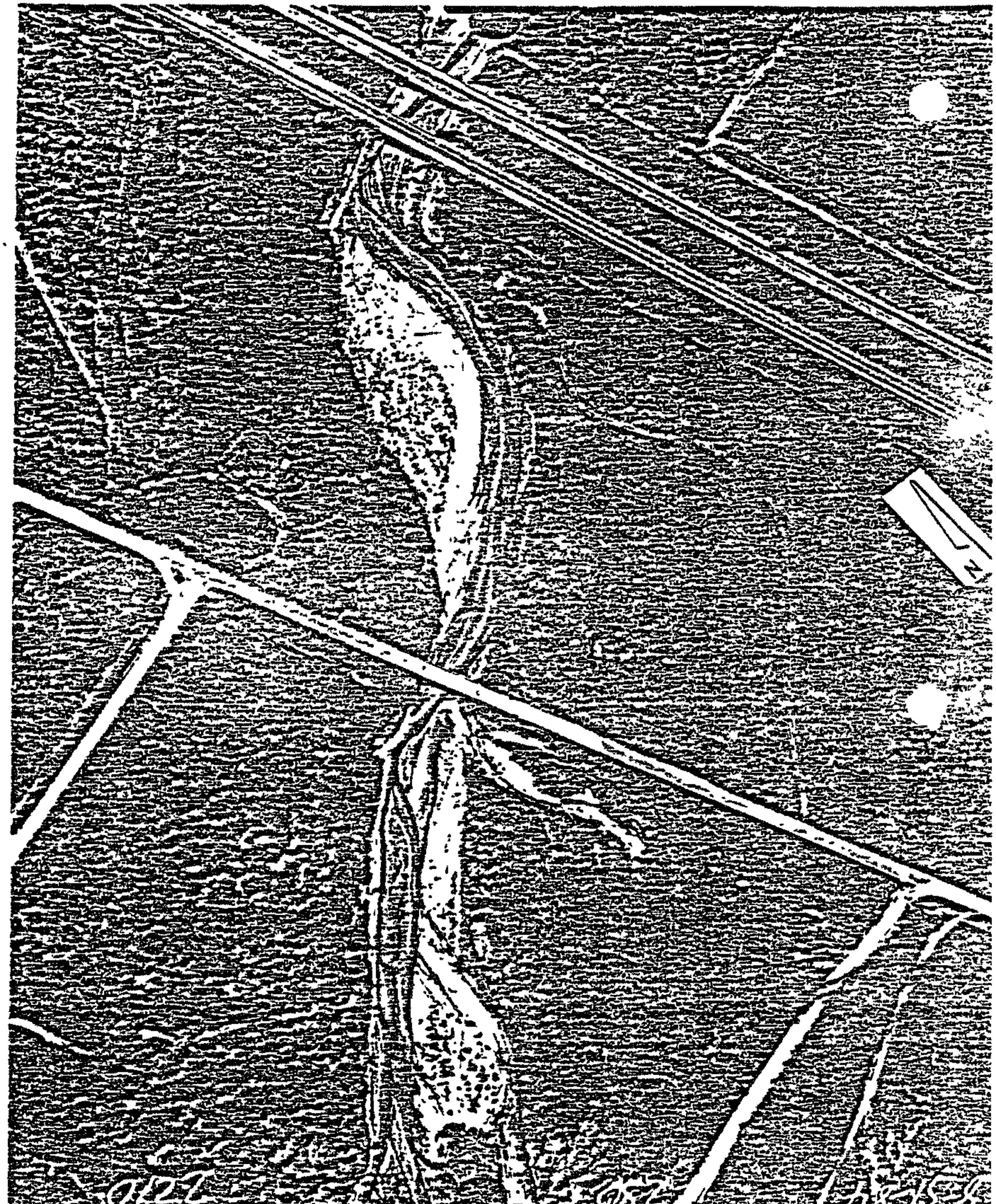
ARIZONA DEPARTMENT OF TRANSPORTATION
HIGHWAYS DIVISION
STRUCTURES SECTION

Sta. 630 +
AGUA FRIA CANYON BRIDGE
SCOUR PROTECTION DETAILS

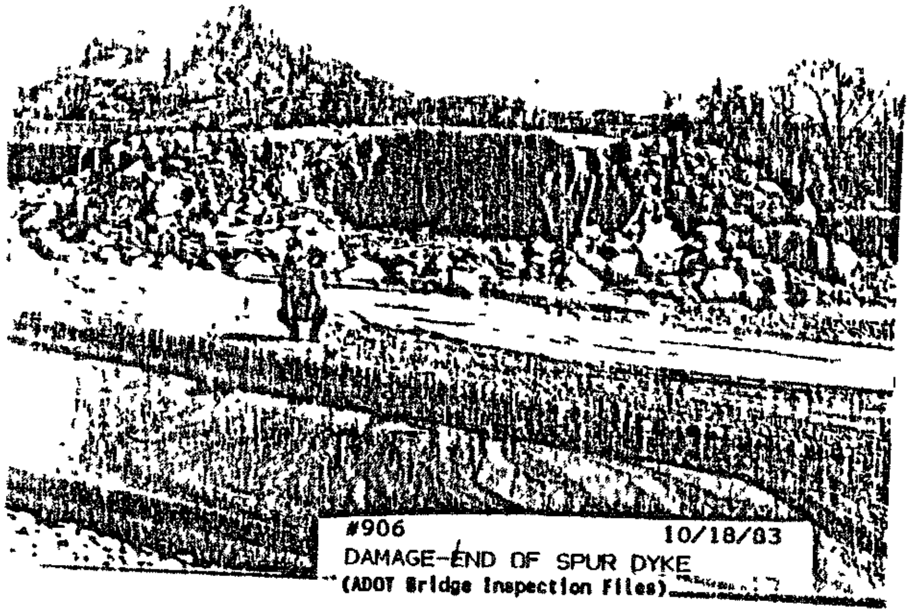
B M D 6/81
C H 6/81

3 3 I 19 SnCruz I 19-1-916

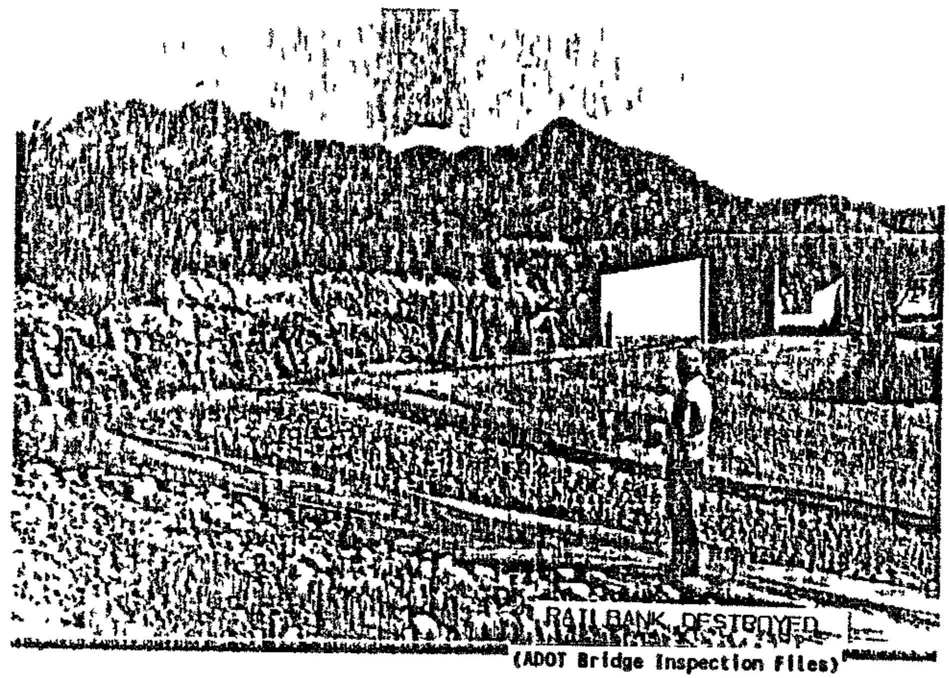
83



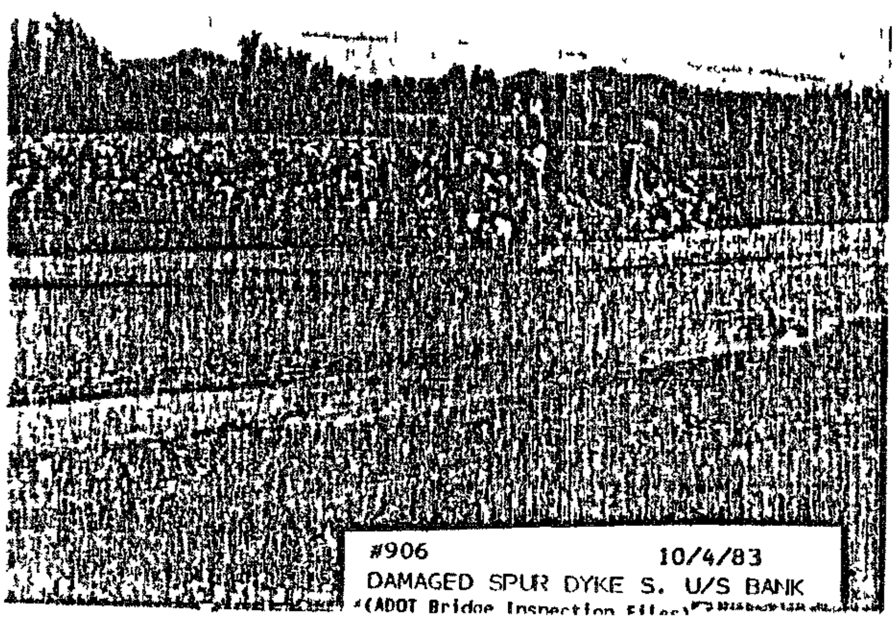
(ADOT Photogrametry Section)



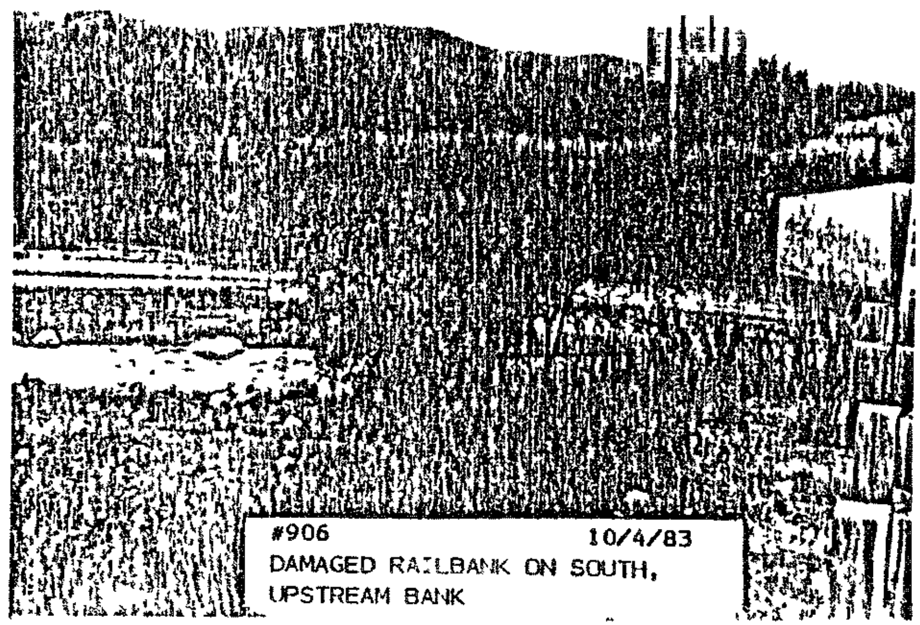
#906 10/18/83
DAMAGE-END OF SPUR DYKE
(ADOT Bridge Inspection Files)



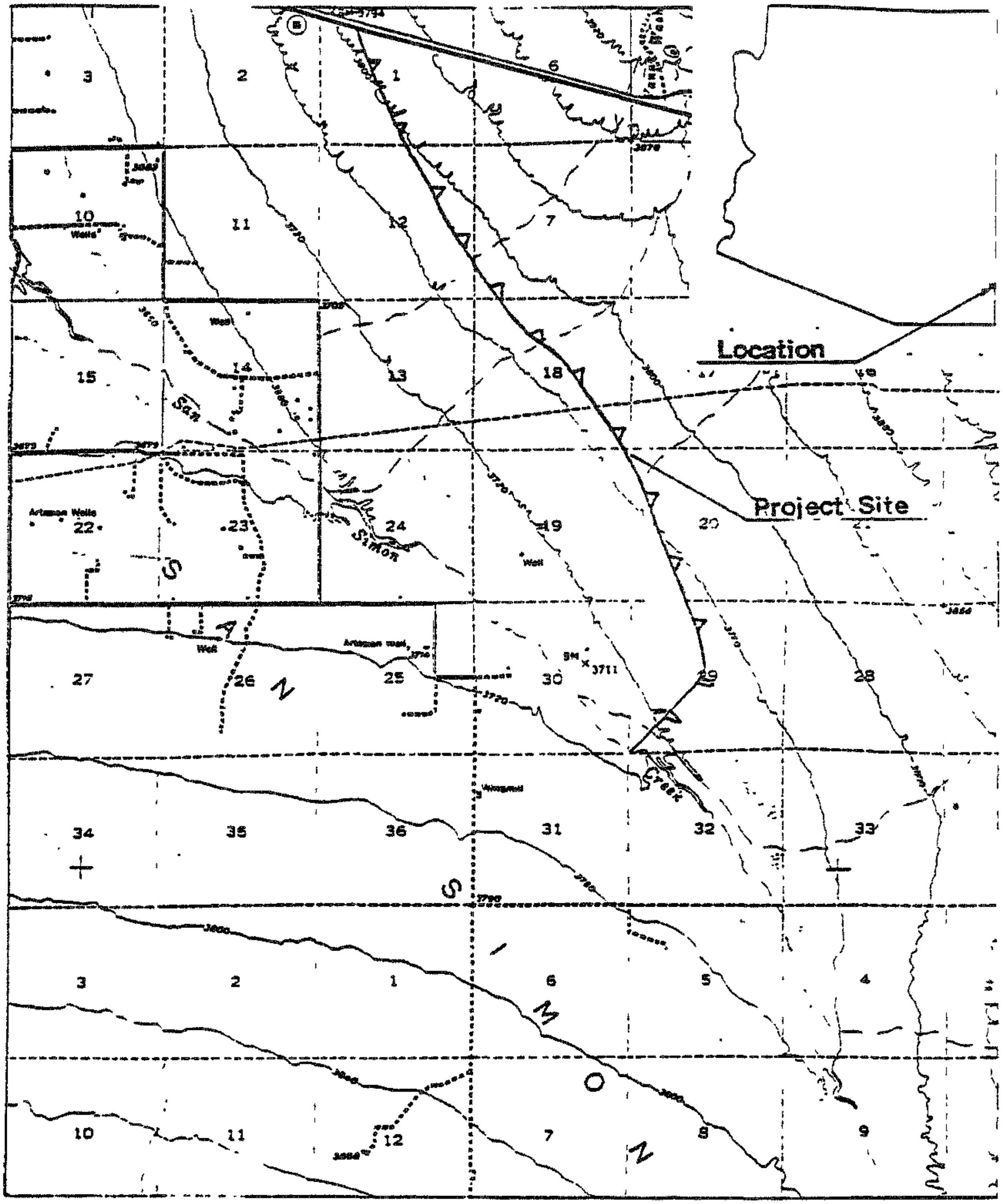
RAILBANK DESTROYED
(ADOT Bridge Inspection Files)



#906 10/4/83
DAMAGED SPUR DYKE S. U/S BANK
(ADOT Bridge Inspection Files)



#906 10/4/83
DAMAGED RAILBANK ON SOUTH,
UPSTREAM BANK



CASE HISTORY NO. 9
Vanar Wash Diversion

Case History No. 9, Vanar Wash

Location:

The diversion is in Cochise County, placed cross slope on an alluvial fan to divert runoff to San Simon Creek. The diversion drains in a south-easterly direction, capturing Vanar Wash, Steins Creek and two other washes.

Type of structure:

The total length of the diversion is 5.64 miles. 4.64 miles consists of a low-flow earth channel excavated with a compacted earth embankment dike. The initial channel gradient ranges from 0.15% to 0.2%. Near Vanar Wash, a 542 foot reach of the dike to protect with riprap from attack by Vanar Wash inflows. Two chute reaches required extensive riprap protection on the invert and side-slopes due to steep gradients of 1.0%. The upper chute has four side-channel inlet structures to allow surface water entry.

Hydrologic characteristics:

The total watershed area is 74.9 square miles. The diversion was designed for a 25-year event with a design discharge of 7,600 cfs and 10,000 cfs in the upper and lower chutes, respectively. Several gauges were built with the project including three in the lower chute. Because the gauges were damaged and did not operate during the storm of August 1971, slope area measurements were used to determine peak discharges and velocities. Roughness coefficients of 0.020 and 0.045 were used for the earth and rock sections, respectively. The estimated peak flow in the upper chute was 7,660 cfs of which 1230 cfs was over the left bank. This left a 6,430 cfs flow in the channel. This slightly exceeded the 25-year design discharge of 7,600 cfs for the upper chute. The estimated peak Q in the lower chute including additional inflows was 8,230 cfs, which is 82% of the 25-year design discharge of 10,000 cfs.

Hydraulic characteristics:

The following table summarizes the hydraulic conditions in the chutes as estimated for design and during the August 1971 flood.

	Design	1971 Flood
Discharge (cfs):		
Upper chute	7,600	7,600
Lower chute	10,000	8,230
Roughness	0.035	0.045
Slope (ft/ft)	0.01	0.01
Side-slope	3:1	3:1

	Design	1971 Flood
Bottom width (ft):		
Upper chute	100	100
Lower chute	120	120
Depth:		
Upper chute	5.3	5.7
Lower chute	5.8	5.9
Froude Number:		
Upper chute	0.96	0.74
Lower chute	0.98	0.78
Velocity:		
Upper chute	12.2	9.4
Lower chute	12.5	10.1

The upper chute has two bends located at the one-quarter and the three-quarter points of the chute. The upper bend was constructed with a radius of 1000 feet and a length of 290 feet, and the lower bend was constructed with a radius of 600 feet and a length of 290 feet. The middle half of the upper chute is straight for 1018 feet at 1.0% gradient. The lower bend was the only bend constructed with a raised bank. The lower chute was constructed with one bend at the top of the chute that accommodated a grade change from .15% to 1.0% in the middle of the bend. The remainder of the chute is straight for 1790 feet at a grade of 1.0%.

Riprap:

The design thickness was specified at 21 inches on the channel bottom with the banks tapering from 21 inches at the base to 17 inches at the top. The as-built plans show the average depth of rock on the bottom was about 5 inches less than the design specified. A post flood measurement at the top of the chute in undisturbed areas show riprap thickness average 14.3 inches and 13.3 inches for the left and right banks, respectively. About 3-5 inches less than the design specified. No thickness changes were specified in the original design for the bends or transitions.

Gradation:

The design calls for a D_{75} of 14.4 inches with calculations based on a straight alignment. The rock size was not adjusted for the curved sections. The tests of gradation during construction gave an average D_{75} of 12 inches. The post flood tests conducted on undisturbed areas gave an average D_{75} of eight inches.

Performance:

Significant damage occurred to riprap in the chute reaches of the diversion channel. Maps of zone of scour and deposition of riprap was prepared by the SCS after the August 1971 flood. The tight radius of curvature and the near critical flow conditions were found to result in riprap stability problems. Problems with riprap gradation and thickness also created performance deficiencies.

VANAR WASH WATERSHED

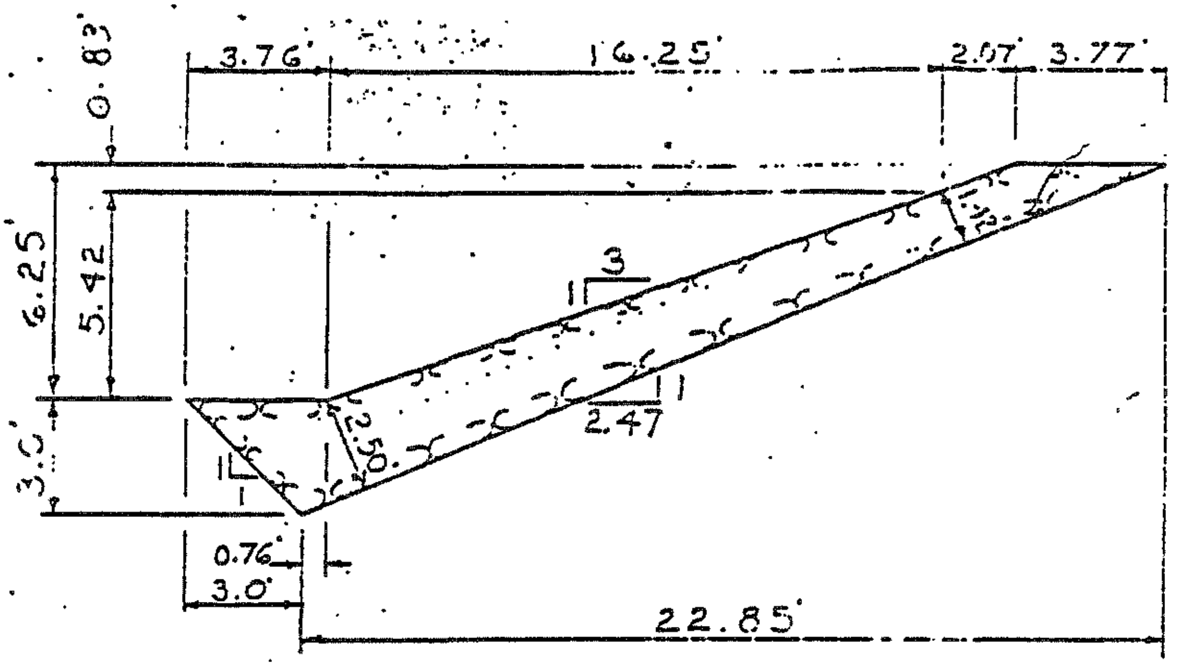
Deficiency Investigation

Rock Riprap Sieve Analysis
Composite of Post Flood Tests Nos. 1-6

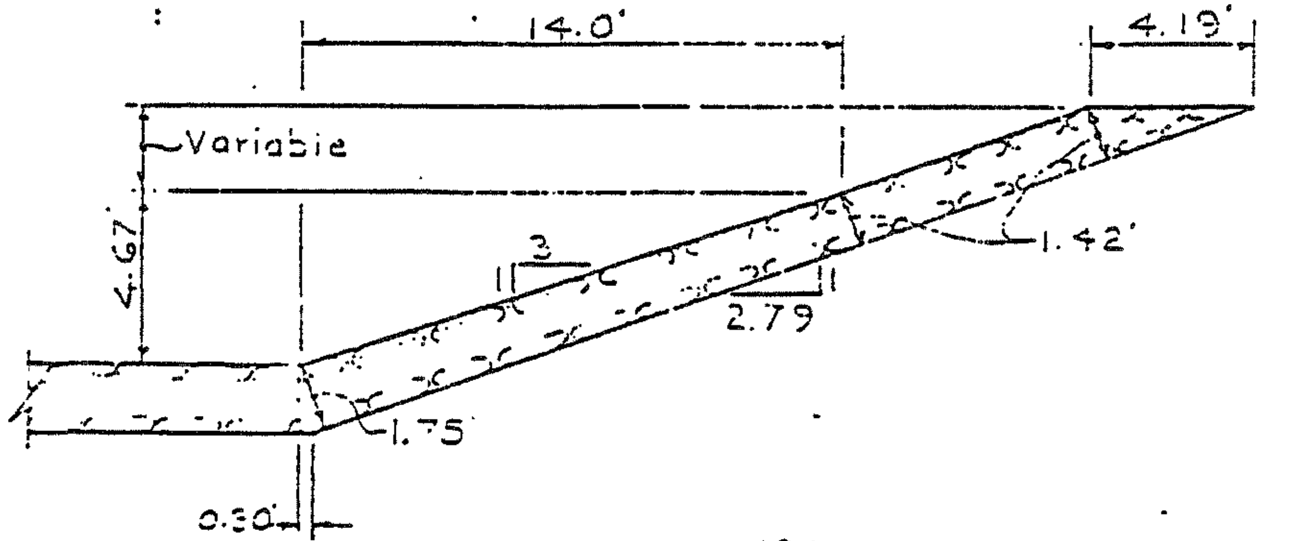
Sieve Size	Weight Retained 1/	Percent Retained	Percent Passing	Percent Passing Required
17"	0.00	0.00	100.00	100
14"	0.00	0.00	100.00	65-90
10"	727.72	11.47	88.53	45-70
6"	1581.11	24.92	63.61	25-50
3"	3239.28	51.06	12.55	10-30
1½"	643.51	10.14	2.41	0-10
Pan	152.88	2.41	0.00	---
Totals	6344.50	100.00		

1/ Totals from tests Nos. 1-6

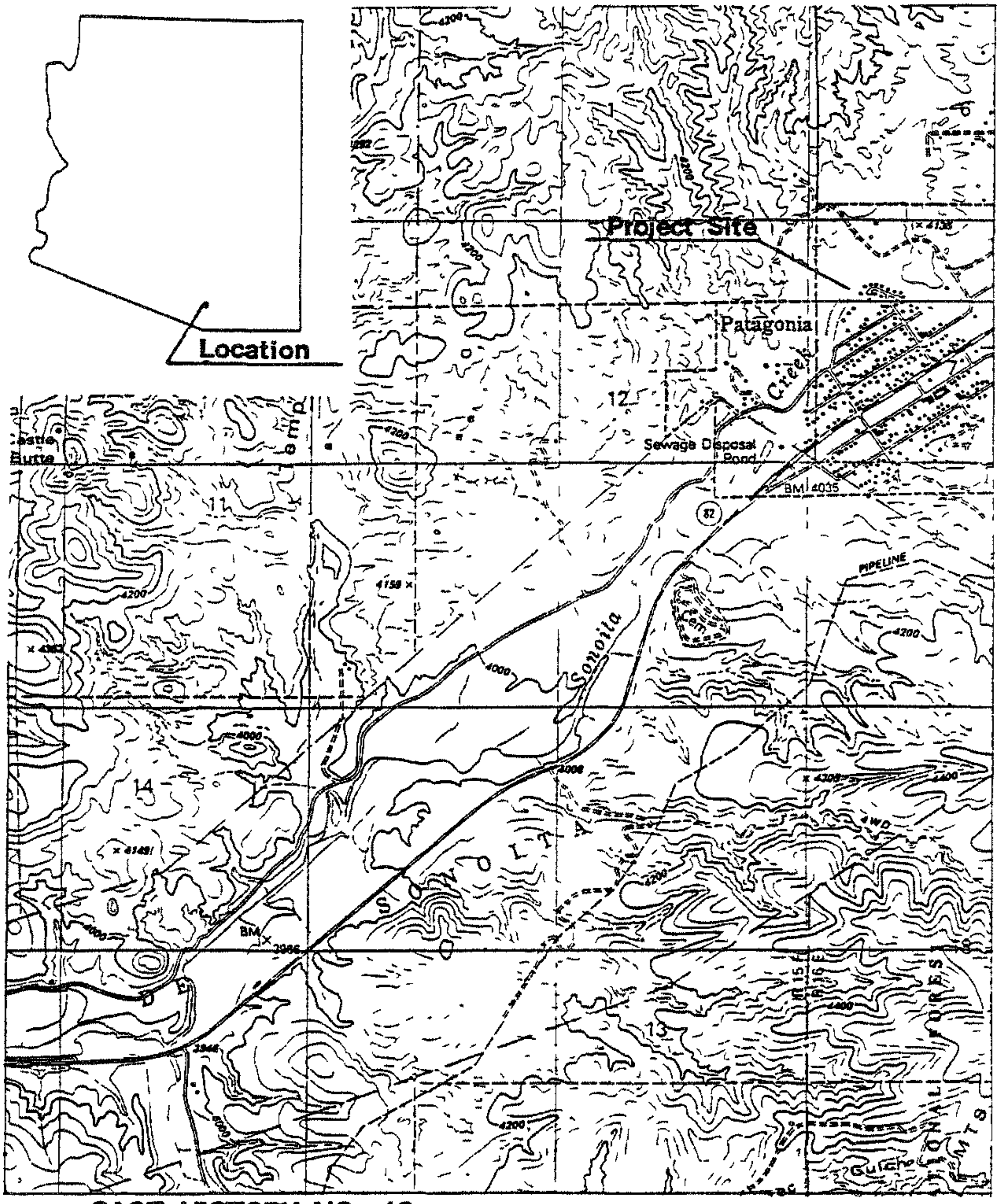
Test Ds = 8" ±
Design Ds = 14.4"



TYPICAL SECTION Sta. ~~93+00 to 103+29.~~
97+00 to 103+49.3



TYPICAL SECTION Sta. ~~196+95.12 to 216+00.0~~
196+06.32 to 215+10
AND Sta. ~~276+44.87 to Sta. 298+20.95~~
274+44.81 to Sta 297+60



CASE HISTORY NO. 10
Sonoita Creek
near Patagonia

Case History No. 10, Sonoita Creek at Patagonia

Location:

Just upstream of U.S. Highway 82 on Sonoita Creek in the Town of Patagonia, Santa Cruz County, Arizona.

Type of Structure:

Rock riprap bank protection along bends of Sonoita Creek.

Hydrologic characteristics:

Gauge #4815 is approximately two miles downstream of Patagonia. The gauge was discontinued in 1972.

General history: 1979

A flood occurred of unknown magnitude instigating the 1982 bank protection work.

1982:

Rock riprap bank protection was installed at three sites, on bank sections that were subject to erosive flow due to a bend in the channel. Site 1 was located just upstream of the bend, on the south bank of the channel. This section was designed to protect a residential area from flow that impinges on the bank as a result a severe bend (radius is 400 feet) 500 feet upstream. The length of Site 1 protection was 600 feet.

Site 2 was located along the outside of the severe bend radius of 200-250 feet, with a topwidth of 80 feet (radius/topwidth of 2.5 to 3.0) just downstream of Site 1. Riprap size was varied along the 710 feet of this installation with the downstream 427 feet specified at a D_{50} of 30 inches and the upper portion specified at a D_{50} of 24 inches. Site 3 was constructed on the outside bank of a mild bend downstream of Site 2. It was constructed to be 245 feet long and protected a road from direct flow impingement. The bend has about a 700-foot radius.

The general specifications for riprap at all three sites as follows:

D_{50} = 24 inches

Side-slope = 3:1

Thickness:

Top of bank = 3 feet (3.75 feet)*

Toe of slope = 6 feet (7.50 feet)*

Toe trench width = 10 feet

Toe down = 4-5 feet

*lower portion of Site 2

The riprap was placed on filter fabric and cutoff walls were constructed at the end of each site. The cutoff walls were six feet thick and extended six feet into the top of the channel bank. The height of the riprap bank protection varied from six-eight feet. The bottom width of Sonoita Creek varies from 40-80 feet in this reach. The channel gradient is 0.69% for Sites 1 and 2, steepening to 0.92% at Site 3.

1983:

A flood occurred, magnitude unknown, that damaged much of the 1982 bank protection.

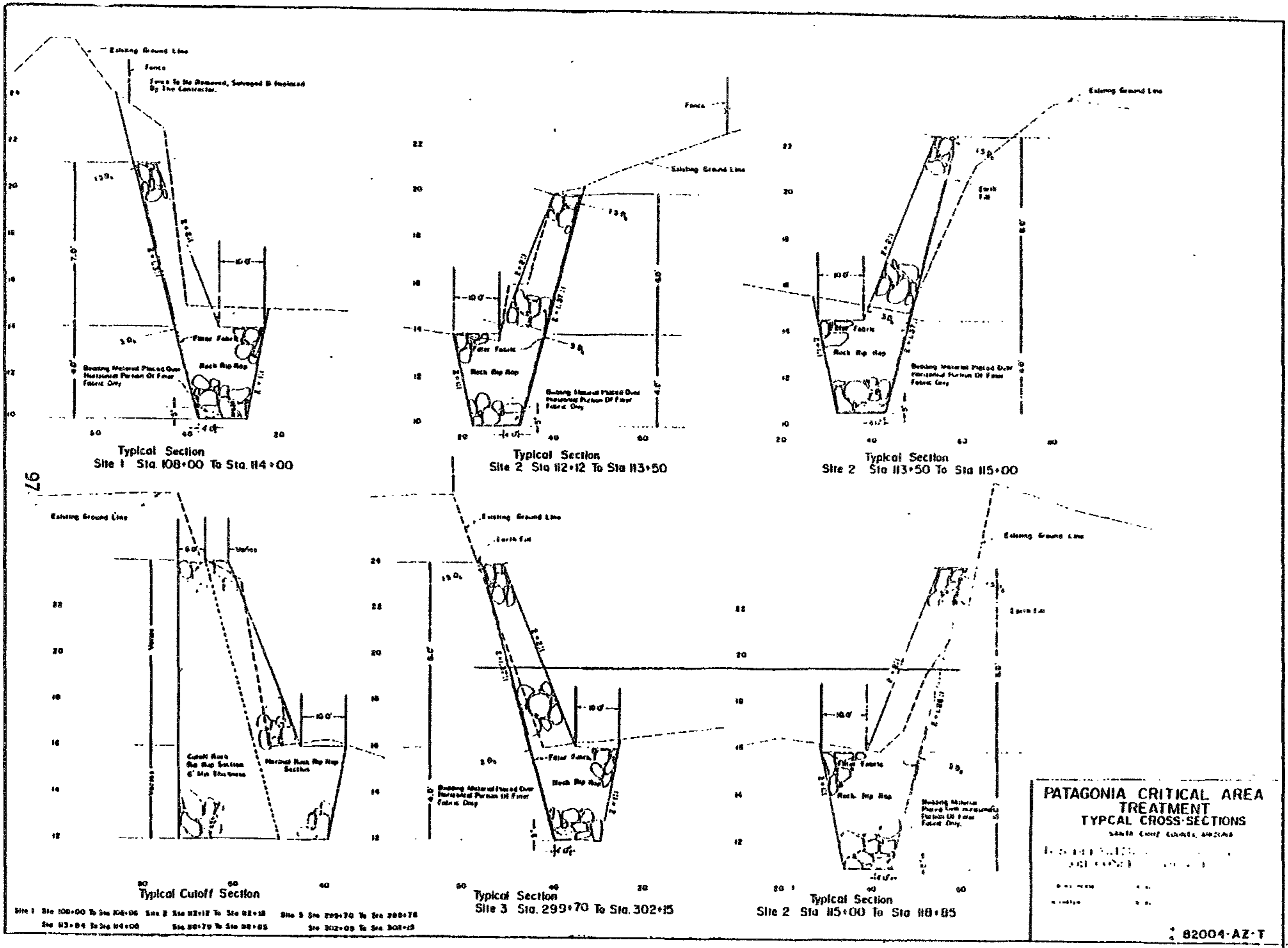
1984:

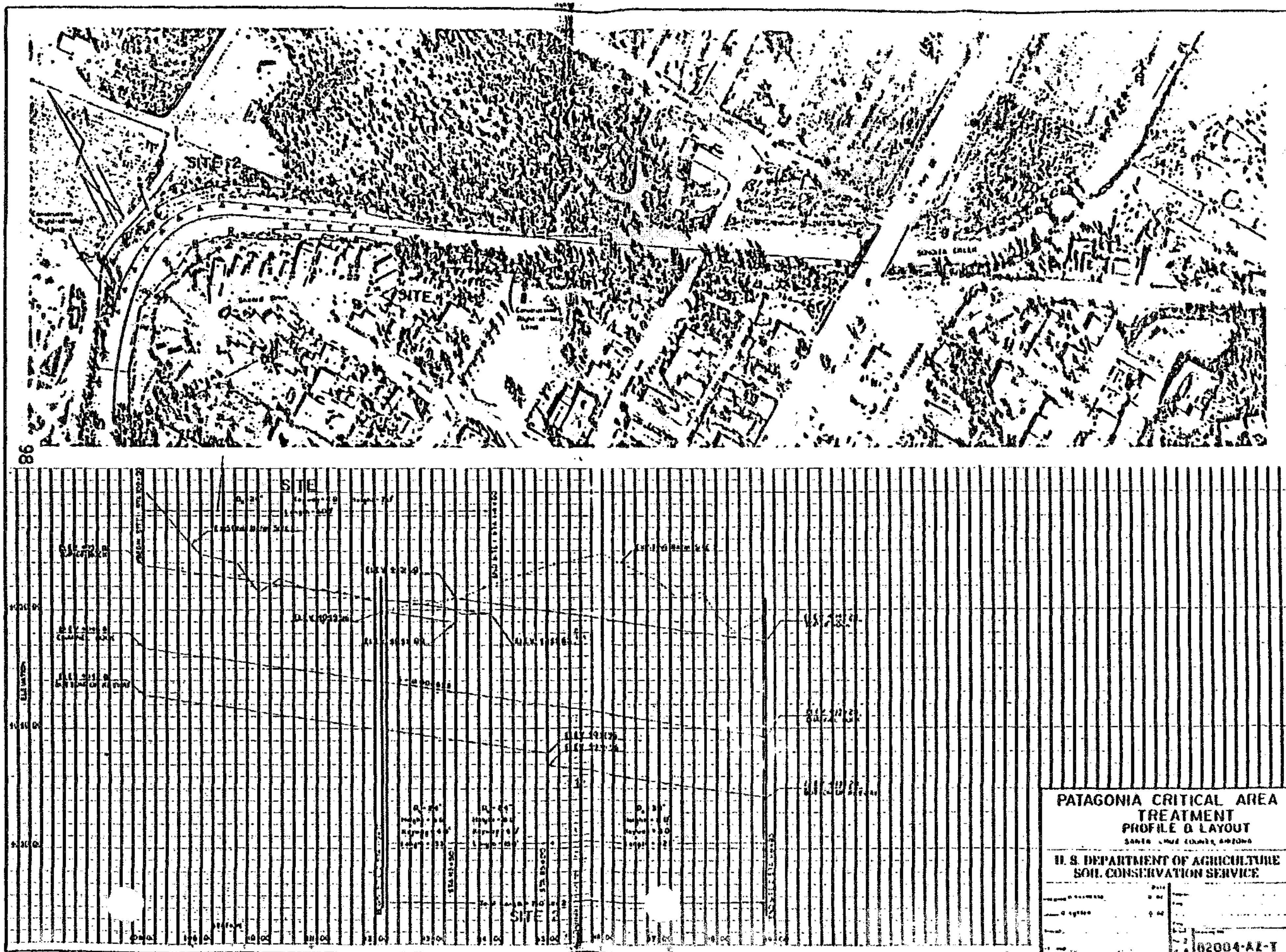
March. The SCS reported that rock-riprap-bank protection installed after the 1979 flood was damaged and rock removed by the 1983 flood. The major damage occurred where filter fabric and the rock were placed over a hard, impervious caliche-cemented formation. a resident observed the failure and reported that the rock was functioning well until the water level was receding. At this time, water appeared to have flowed between the caliche formation and the filter fabric, the fabric ballooned causing the rock to roll downslope. The SCS considered this to be a plausible cause of the failure, because of the hydrostatic force that would occur with the water trapped between the impervious caliche and the low permeability filter fabric.

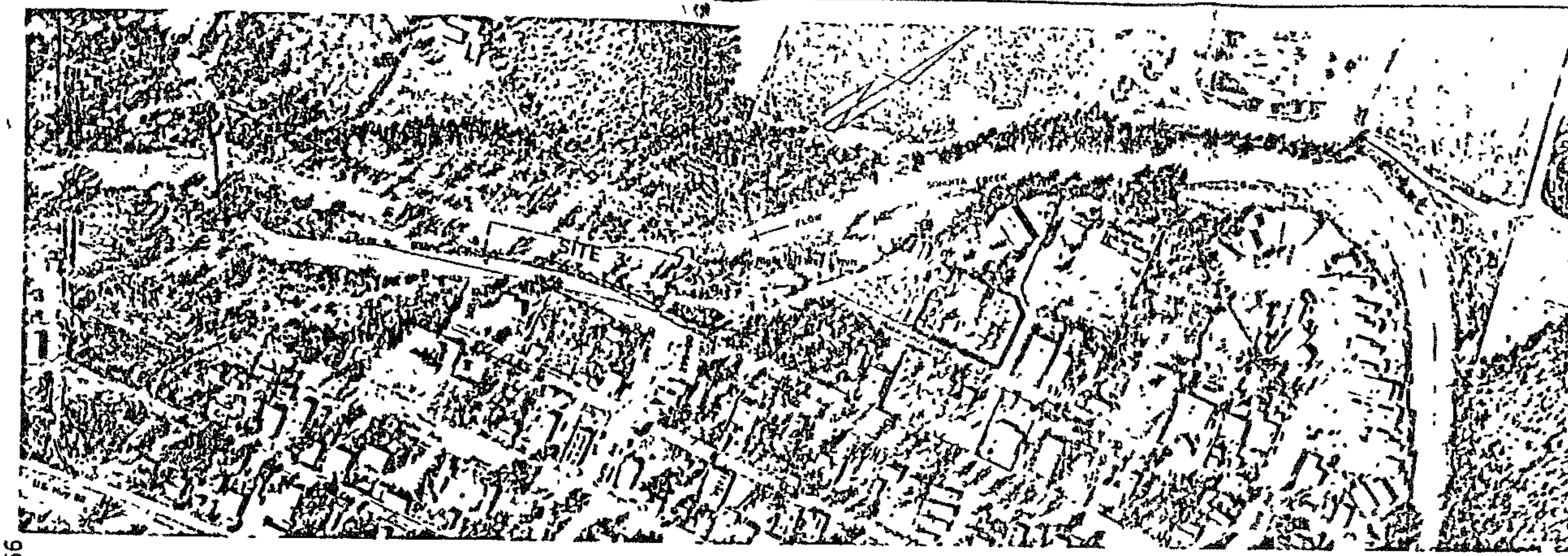
Rock riprap bank protection was repaired on Sites 1 and 2 of the 1983 construction.

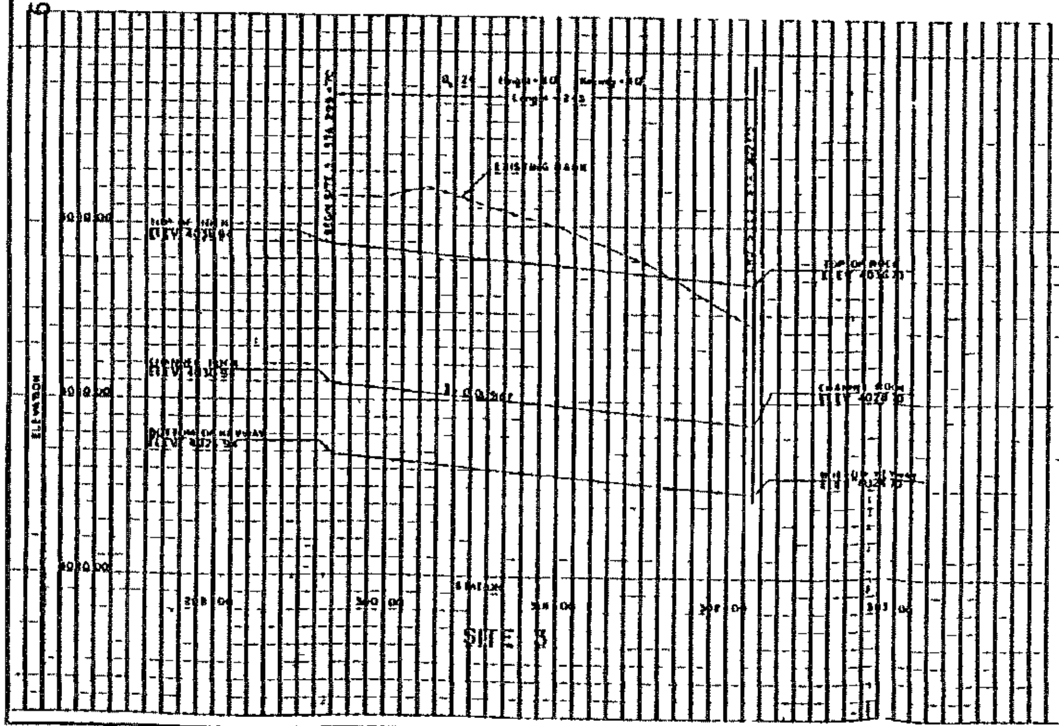
Site 1: 127 feet of bank was replaced in the lower third of the original bank repair area. The replacement bank section was the same as the one used in the 1982 construction. No new specification for rock riprap size or gradation was given. It was reported that about 1.5 feet of silt deposited on the bottom but the riprap from the silt level up was totally removed, with about 5-10 feet of natural embankment behind the original riprap washed out. The toedown area was reported to have not been disturbed.

Site 2: 185 feet of bank is replaced in the lower quarter of the original bank-repair area with an additional 100 feet of bank protection placed beyond the end of the original Site 2 work. This area was specified to be built with D_{50} of 30-inch rock, and the replacement bank section was constructed to this specification. The damage to the bank was reported to be very similar to the damage at Site 1. About one foot of silt deposited on the bottom and above that all the rock was removed with up to three feet of scour into the natural bank behind the riprap. The toedown portion was not disturbed.

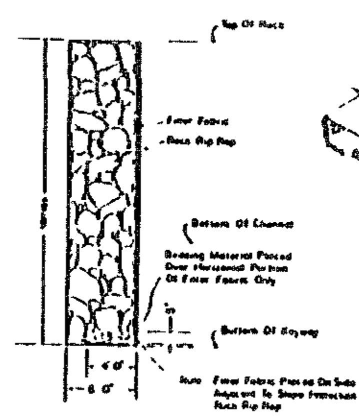

Typical Section
Site 1 Sta. 108+00 To Sta. 114+00
Typical Section
Site 2 Sta 112+12 To Sta 113+50
Typical Section
Site 2 Sta 113+50 To Sta 115+00
Typical Cutoff Section
Typical Section
Site 3 Sta. 299+70 To Sta. 302+15
Typical Section
Site 2 Sta 115+00 To Sta 118+85
Existing Ground Line
Fence
Rock Rip Rap
Filter Fabric
PATAGONIA CRITICAL AREA
TREATMENT
TYPCAL CROSS-SECTIONS
82004-AZ-T


SITE 2
SITE 1
SITE
PATAGONIA CRITICAL AREA TREATMENT
PROFILE & LAYOUT
SANTA CRUZ COUNTY, ARIZONA
U. S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE
62004-AZ-T

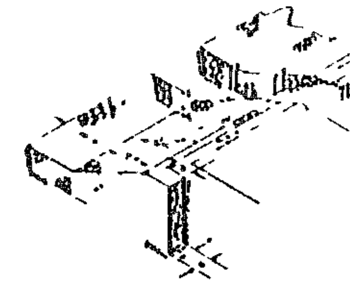




SITE 3



TYPICAL CUTOFF
Not To Scale

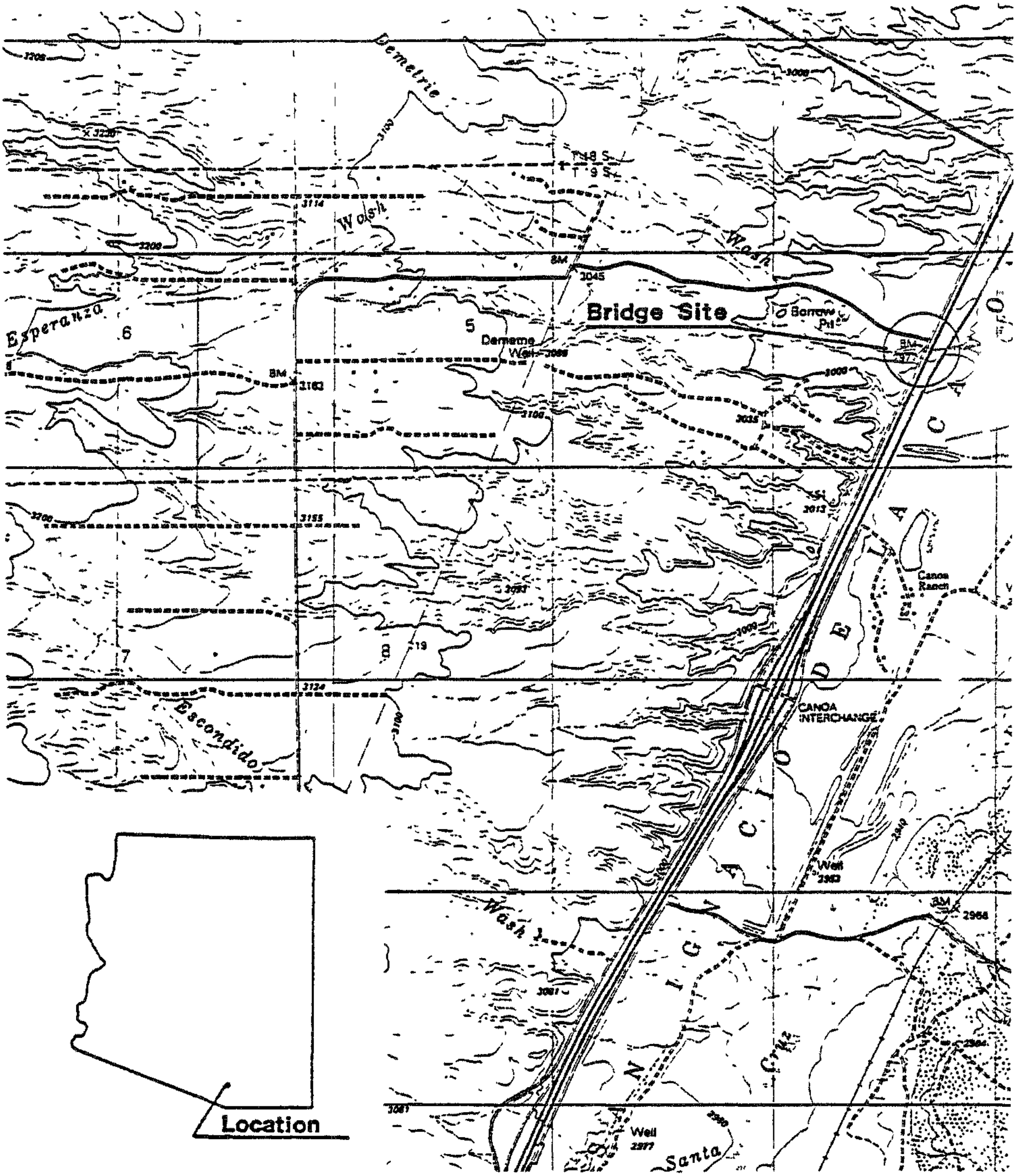


TYPICAL CUTOFF
ISOMETRIC VIEW
Not To Scale

PATAGONIA CRITICAL AREA TREATMENT
PROFILE & LAYOUT
SANTA CRUZ COUNTY, ARIZONA

U.S. DEPARTMENT OF AGRICULTURE
SOIL CONSERVATION SERVICE

82004-A2-T



CASE HISTORY NO. 11

Esperanza Wash

Route: I—19

Case History No. 11, I-19, Esperanza Wash Bridges

Location: Approximately 28 miles south of Tucson

Type of crossing:

Three bridges: northbound, southbound and west frontage road with lengths of 194 feet, 168 feet and 168 feet, respectively. The skew of the crossings are 45°, 35°, and 20° with eight, seven, and seven spans, respectively. All three bridges are continuous reinforced concrete slab bridges.

Hydrologic characteristics:

Drainage area = 40.5 square miles with a Q_{50} = 9500 cfs. The high water mark is estimated at 2967.5 feet. This data was obtained from the "As-Built Plans" of the southbound and west frontage road bridges completed in January 1977. Current designs use Q_{max} = 10,500 cfs with a velocity of 17.2 ft/sec.

Hydraulic characteristics:

This stream would most likely be characterized as a meandering incised alluvial channel. For the 50-year flood frequency, the average depth of flow was nine feet, the Froude number = 0.80, and a theoretical scour estimate of 6.6 feet. The upstream channel has been constricted by dumping of waste concrete on the north channel bank. Four to six feet of degradation had occurred from the ADOT materials pit (#7008) located downstream to the channel checks. This made the adequacy of the spread footings totally dependent on the existence of the channel checks.

Crossing history: 1971

The May 5th inspection of the northbound bridge revealed that the streambed had degraded up to four feet between spans 3 and 7 since the previous inspection on March 18, 1969. The change in stream bed elevation was attributed to the excavation of the downstream ADOT aggregate pit #7008. The streambed continued to degrade from one to four feet under spans 3, 4, and 5 until 1976.

1976:

Channel checks (C-17.02 type 4) were constructed downstream in conjunction with the construction of the southbound and west frontage road bridges. There has only been minor degradation of the stream bed under the southbound and west frontage road bridges since the original construction.

1977:
June 23rd experienced a high water mark of elevation of 2960 feet at the southbound bridge. The estimated flow was 1200 cfs.

1981:
The April 7th inspection indicated the channel checks had some missing rock, damaged wire, and a scour hole below the lower basket.

1982:
In late September, ADOT's Scour Team investigated Esperanza Wash bridges and identified several problems:

1. The integrity of the C-17.02 channel checks and wire-tied aprons were suspect because of corroded and broken wire.
2. The north side of the channel check had no cable lacing between the rails, possibly due to scour damage.

The following recommendations were made: Repair or replace the existing grade control structure, and continue to refrain from excavating below the streambed in pit #7008.

1983:
On October 1st, the crossing experienced another flood. The dumped riprap bank protection was not damaged at the north and south abutments of the northbound bridge. The downstream check structure was intact with five feet + scour at the downstream side of the second check; apron remained intact. Railbank at both ends of check structure were heavily damaged with accompanying extensive bank erosion at the north end. New end treatment for the check structure and northbound bridge were needed.

1984:
Esperanza Wash was one of a number of sites studied in the March backhoe investigation by ADOT. Two trenches were excavated, one halfway between the

west frontage road and the southbound bridge along the north bank, and the second approximately ten feet upstream of the west frontage road bridge along the north bank. The first trench revealed no evidence that riprap had ever been placed while the second trench found the riprap toe trench intact about three to four feet below the channel. Approximately 130 feet of the riprap on the north bank was damaged from the southbound bridge to the east/west R.O.W. fence upstream of the west frontage road bridge. Estimated 95% of dumped riprap on north bank was gone.

The following recommendation was presented with the results of the backhoe investigation: Regrade and grout from the centerline of the southbound bridge to the railbank protection anticipated for new spur dike construction.

Riprap design:
Scour was determined using Lacey's regime formula:

$$d_m = 0.47 \left(\frac{Q}{f} \right)^{1/3} = \text{mean depth}$$

It was conservatively determined that the mean depth = 8.2 feet resulting in a scour depth of 4.8 feet. The riprap design equation developed by Ray Jordan, head of ADOT hydraulics group at that time was used:

$$D_{50} = 0.0006086 \frac{V^3}{Y^{\frac{1}{2}}} (K_{sf}) (K_{50}) (K_{ss}) (K_b)$$

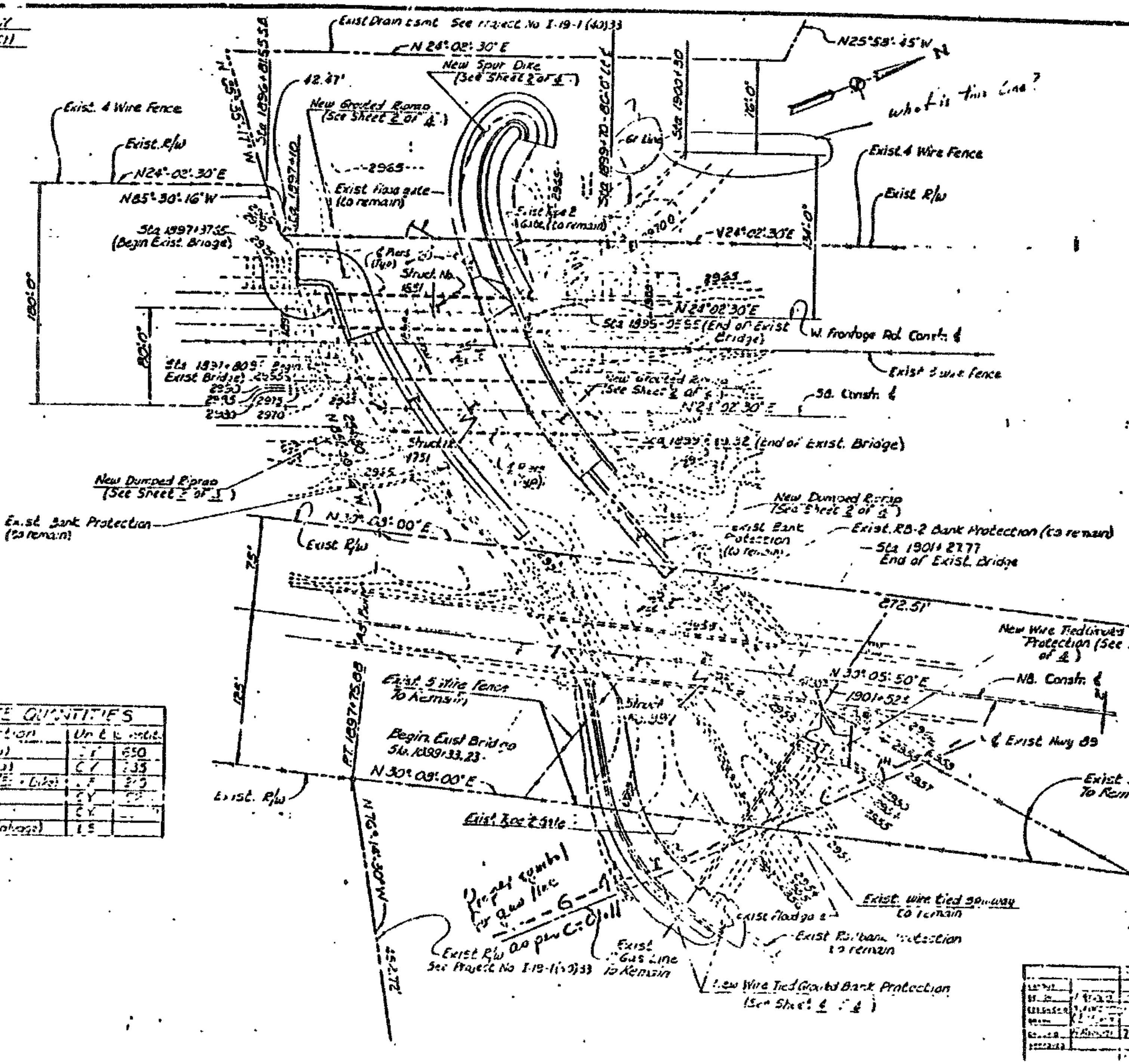
It was determined that a D_{50} = 2.14 feet was needed by this method. In contrast, the design was checked using HEC-15 which predicted a D_{50} = 1.6 feet. It was decided to use D_{50} = 24" since that is what was currently in place.

The following table is the gradation specified for dumped and grouted riprap:

Percent Passing	Rock Size	
100	18"	
40-60	12"	minimum specific gravity = 2.4
20-40	18"	
0-5	6"	

The following table is the gradation specified for riprap in the toe trench:

Percent Passing	Rock Size	
100	36"	
40-60	24"	minimum specific gravity = 2.4
20-40	18"	
0-5	6"	

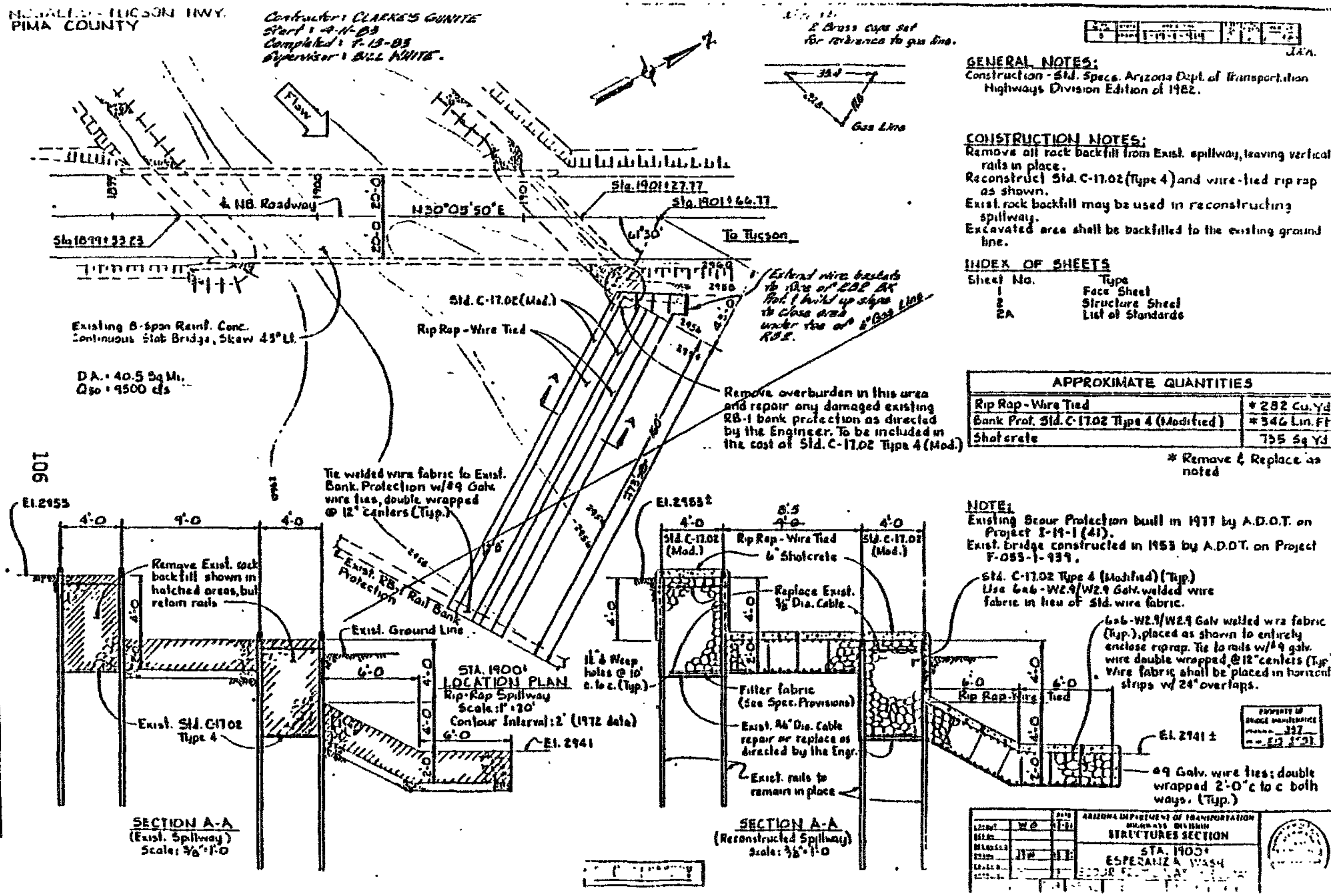


GENERAL NOTES:

1. Construction- Arizona Depart. transportation Standard for Road and Bridge Const Edition of 1982, revised to
2. For wire tied bank protection Rock shall be sound and durable or angular shape and with diameter of 8" min and Flat or needle shapes are not Rock shall be comprised of 12" and 50% max 18" to
3. For dumped and grouted riprap Rock shall be sound and durable or angular shape and with average diameter (D_{50}) Flat or needle shapes are not
4. Specific Gravity of rock sha
5. Remnants of existing bank be removed by the Contr required for new constr disposed of as directed Engineer
6. It is the responsibility of the that consideration be give safety of live stock in area
7. Contractor to salvage exist gates as directed by the
8. Dimensions shall not be scaled drawings
9. Contour elevations shown were time of original constr bridge, but may not be so time.

ESTIMATE QUANTITIES

Description	Unit	Quantity
[illegible]	[illegible]	650
[illegible]	CY	135
[illegible]	[illegible]	[illegible]
[illegible]	CY	[illegible]
[illegible]	CY	[illegible]
2 Gates (Salvage)	LS	



GENERAL NOTES:
Construction - Std. Specs. Arizona Dept. of Transportation Highways Division Edition of 1982.

CONSTRUCTION NOTES:
Remove all rock backfill from Exist. spillway, leaving vertical rails in place.
Reconstruct Std. C-17.02 (Type 4) and wire-tied rip rap as shown.
Exist. rock backfill may be used in reconstructing spillway.
Excavated area shall be backfilled to the existing ground line.

INDEX OF SHEETS

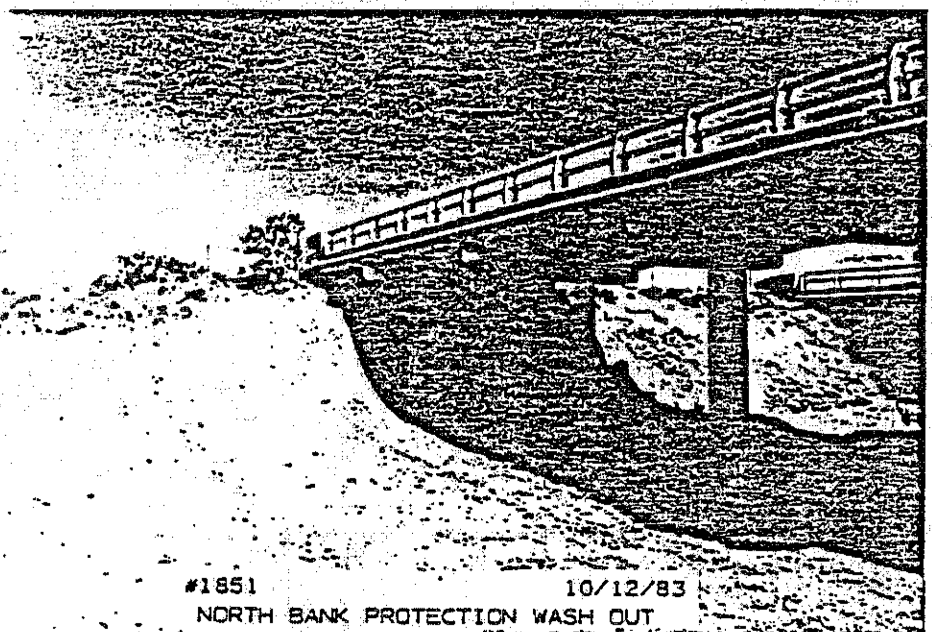
Sheet No.	Type
1	Face Sheet
2	Structure Sheet
2A	List of Standards

APPROXIMATE QUANTITIES

Rip Rap - Wire Tied	* 282 Cu. Yd
Bank Prot. Std. C-17.02 Type 4 (Modified)	* 346 Lin. Ft
Shotcrete	735 Sq. Yd

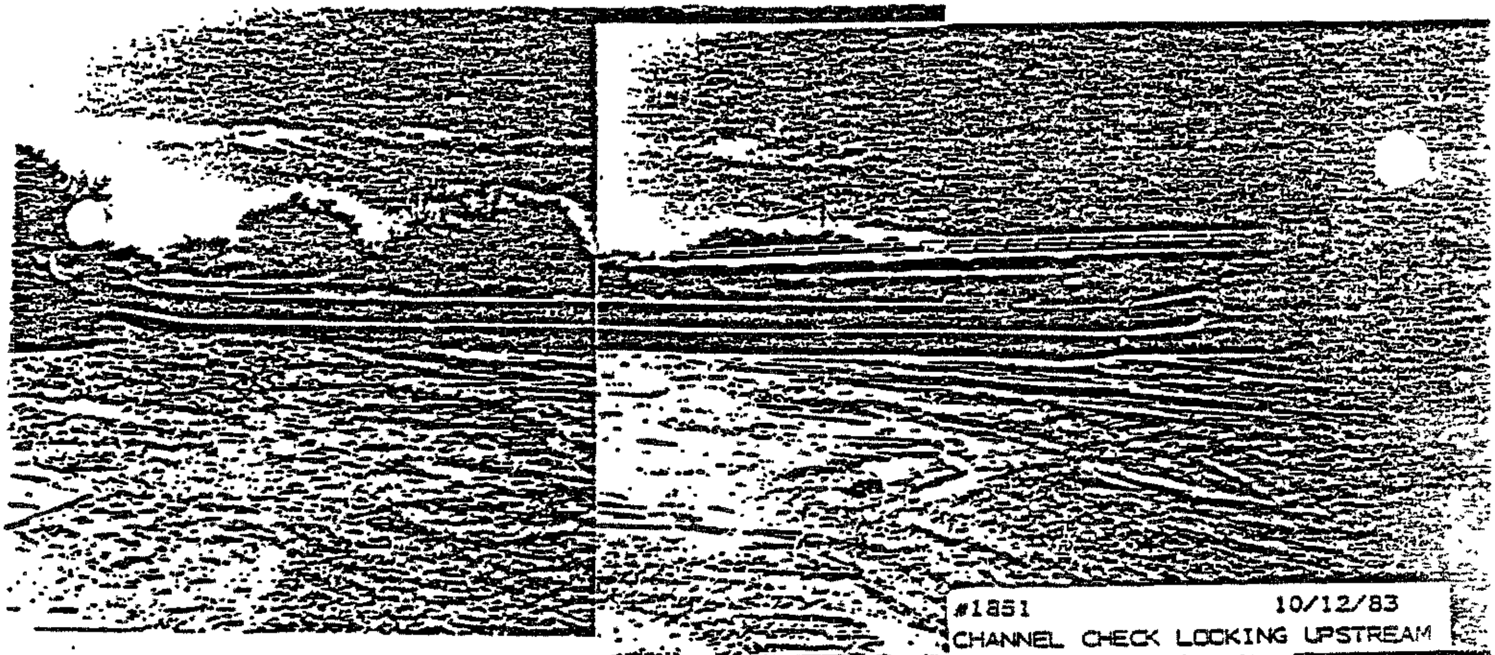
* Remove & Replace as noted

NOTE:
Existing Scour Protection built in 1977 by A.D.O.T. on Project I-19-1(41).
Exist. bridge constructed in 1953 by A.D.O.T. on Project F-053-1-939.

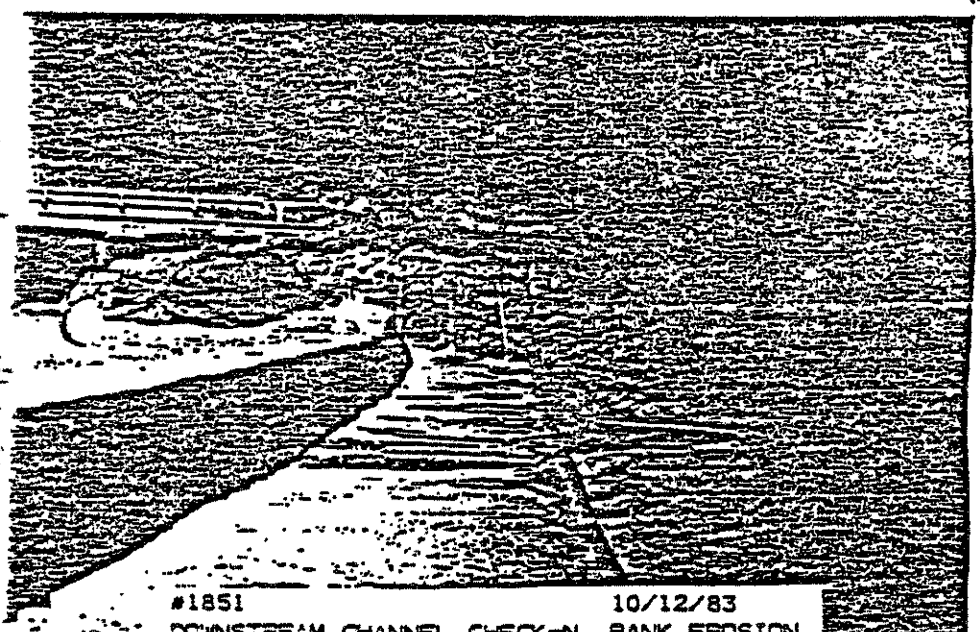


(ADOT Bridge Inspection Files)

(ADOT Bridge Inspection Files)

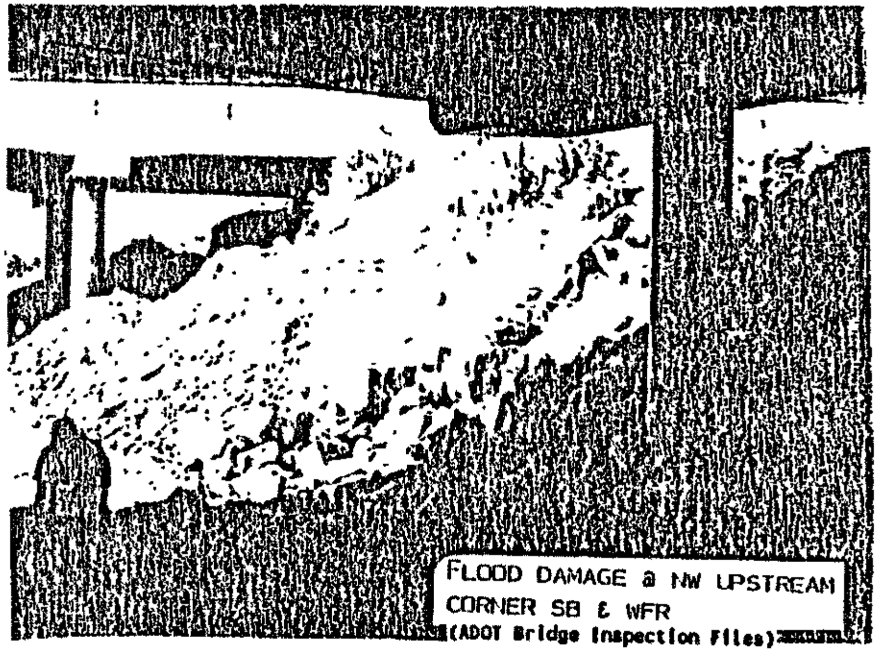


#1851 10/12/83
CHANNEL CHECK LOOKING UPSTREAM
(ADOT Bridge Inspection Files)

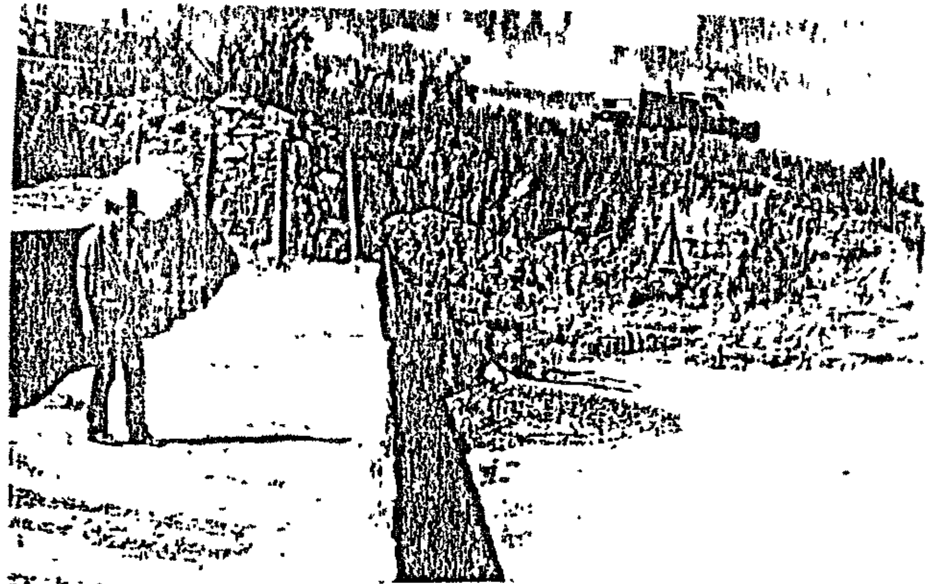


#1851 10/12/83
..DOWNSTREAM CHANNEL CHECK—N. BANK EROSION
(ADOT Bridge Inspection Files)

#1851 (ADOT Bridge Inspection Files) 10/12/83



FLOOD DAMAGE @ NW UPSTREAM CORNER SB & WFR
(ADOT Bridge Inspection Files)



DAMAGE @ NORTH END OF CHECK WALL
(ADOT Bridge Inspection Files)

DAMAGE @ SOUTH END OF CHECK WALL
(ADOT Bridge Inspection Files)

VI. SUMMARY OF FAILURE CONDITIONS

The case histories illustrate the variety of failure conditions possible for riprap protection measures for bridges in Arizona. There are also cases where riprap protection has been designed using conventional methods and has performed in flood conditions near the estimated design discharge. the variation in riprap performance is difficult to assess quantitatively, because measurements of hydraulic conditions, at or near the time the failure occurred, were not made. Even discharge records are limited at the sites because of the large number of gages that were discontinued prior to major floods in 1983.

The causes of riprap failure fall into five general catagories including: riprap quality, riprap layer characteristics, hydraulic conditions, site conditions, and river behavior. Riprap quality addresses material characteristics of rock used and includes the properties of density, durability and shape. Although construction specifications reviewed for case histories were limited regarding quality at all sites, performance problems were not associated with inadequate quality. The Vanar Wash case history indicates that breakage of riprap from the quarry to the site contributed to a reduced riprap size. This may be a more prevalent problem than i. realized, because it is difficult to measure the size and gradation of riprap after it has been installed.

Riprap layer characteristics include the size, gradation and thickness of the installed riprap protection. The Vanar Wash Diversion case history documents problems in attaining required characteristics during construction. Methods that permit effective measurement of these characteristics in the field are lacking. Another important riprap layer characteristic is the underlying bedding layer upon which the riprap is placed. Two types are used: a gravel bedding and a synthetic geotextile fabric. At the Patagonia Wash failure, the SCS reported that the fabric filter contributed to the failure by trapping flow between a relatively impermeable, cemented bank and the fabric filter. The uplift pressure and the fabric strength permitted the riprap layer to be lifted off the bank. This failure was the only known case of a geotextile fabric contributing to a riprap failure. Since failure conditions at the site were not completely investigated by the SCS, it cannot be assumed that many other factors did not contribute to the failure.

Hydraulic conditions result in the boundary shear force on the riprap layer. This boundary shear force is in turn resisted by the weight of the stones comprising the riprap layer. The design methods used at case study sites (primarily HEC-15 and Report 108 by Anderson) use an allowable shear stress on a riprap element equal to four to five times the mean stone size in the gradation. This value is based on flume studies and limited field data. The boundary shear stress has estimated base of the average tractive force. Allowances were not made for many factors that could locally increase the boundary shear stress such as at channel bends and encroachments. Although riprap design information is sketchy, one of the primary reasons for riprap failure or erosion problems is the underestimation of boundary shear stress. In most cases, riprap design was based on the assumption of a uniform, straight channel while in reality other factors that greatly increase the boundary shear stress were present. The case history of bridges on the Santa Cruz River, Rillito River, and the SCS projects at Vanar Wash Diversion and Patagonia Wash show that riprap failures occur when these additional factors are ignored.

Site conditions and river behavior play a significant role in the performance of riprap protection. The site conditions relate to the shape and profile of the river in the vicinity of the bridge at the time the bridge is constructed. The river behavior relates to the change in river form, shape and profile over time. Conditions at bridges in this study show the complexity of these sites. Many of the sites are located in or near channel bends including bridges on the New River (I-17), Santa Cruz River (I-19), Rillito Creek (I-10), Old Junction Wash (I-19), Agua Fria Canyon (I-19), Esperanza Wash (I-19), Vanar Wash Diversions, and Sonoita Creek. In some of these cases, migration of the channel bend has gradually changed the severity of the bend at bridge site. At bridges on Rillito Creek (I-10), Old Junction Wash (I-19), Agua Fria Canyon (I-19), and Esperanza Wash (I-19), this has lead to the construction of control measures in the reach upstream of the bridge to direct the flow at a less severe approach angle to the waterway opening.

Other river stability problems that were found in this study include region channel bed lowering (Granite Creek, US 89), and channel widening (Tinaja Wash, I-19 and Santa Cruz River, I-19). Some problems are the result of man's activity in the river system. The region channel bed lowering at Granite Creek is the consequence of large scale sand and gravel mining downstream of the bridge site.

Table I summarizes hydraulic conditions and riprap failure causes at the bridge sites reviewed in this study. These case histories demonstrate both the variety and complexity of river channel conditions at bridge crossings and the resulting difficulties in establishing riprap protection at critical areas associated with the bridge structure. The histories also show the limitations of riprap design methodologies and to some extent hydraulic modeling.

Riprap design procedures employed have limited or no ability to address special conditions found at bridge crossings. At a minimum, a riprap procedure should address areas of increased boundary shear stress found in channel bends and zones of accelerating flow at spur dikes and abutments. Flow zones with vortices such as piers and areas of expanding flow are less tractable problems. Few models of bridge hydraulics provide output other than average hydraulic conditions. Procedures for determining the actual distribution of boundary shear stress at bridge site are lacking. Since the integrity of the bridge crossing depends to a large extent on performance of protective critical area at the site, the lack of procedures addressing the distribution of boundary shear stress at bridge waterway appears to be a critical gap in riprap design.

TABLE 1. SUMMARY OF HYDRAULIC CONDITIONS AND RIPRAP PROTECTION PERFORMANCE

SITE NAME DRAINAGE AREA D_{50} SIZE	FLOOD DATE	Q (cfs)	V (fps)	(F) FROUDE NUMBER	DEPTH (ft)	WIDTH (ft)	(R) RADIUS OF CURVATURE (ft)	D_{50}	SLOPE	To lb/ft^2	CAUSE OF FAILURE	COMMENTS
I-19, Santa Cruz River	1967	5800 (USGS)									Particle erosion @ embankment.	During construction flow scoured sheet piling.
D.A.=2070 sq mi	1977	22000 (USGS)	13.7	0.74*	10.6	152*			0.0029	1.92	River migration, particle erosion @ embankment, dike and bend.	Extensive bank protection damage @ most areas of the crossing.
D_{50}=12" (1978) D_{50}=18" (1984)	1978	13500 (USGS)							0.0037	2.77	Particle erosion @ embankment.	Failure of sheet piling and rock protection.
	1983 (Feb)	5000 (USGS)									Particle erosion @ embankment and bend.	Heavy rubble replaced missing riprap during flow, 150' of riprap lost.
	1983 (Oct) Design Flood	45000 (ADOT)	11.7	0.60*	12.0	321*	360	18"	0.0114	2.42	All phases of bank slumping and particle erosion.	Flow was much larger than ever anticipated. No protection designed for this magnitude.
I-17, Dead Man Wash	1959	1850										
D.A.=12.4 sq mi	1978	1400										
D_{50}=22" (1979)	1979										Scour of channel bottom.	Scour within 2 feet of top of pier footings. Riprap invert and embankment placed.
	Design Flood (Q_{50})	8800	10.63*	1.10	2.9	300*	NA		0.0085	1.54		Current inspections report little flow or alteration to riprap.
I-17, New River Bridges	1967	12000										
D.A.=82 sq mi D_{50}=15" (1968)	1968	NA										Bridges were constructed and river was channelized with riprap banks and boulders.
	1970	19500										Channel degradation under bridge.
	1978	17900										Riprap was dumped along both banks 4' above observed H.W. mark.

I-17, New River continued on next page

* Indicates calculated values

TABLE I (continued)

SITE NAME DRAINAGE AREA D_{50} SIZE	FLOOD DATE	Q (cfs)	V (fps)	(F) FROUDE NUMBER	DEPTH (ft)	WIDTH (ft)	(Rc) RADIUS OF CURVATURE (ft)	D_{50}	SLOPE	To lb/ft^2	CAUSE OF FAILURE	COMMENTS
I-17, New River continued...												
D_{50}=38" (1979)	1979	4600									Particle erosion of embankment.	The dumped riprap failed near deflection wall. Riprap invert and embankment constructed.
	1980	"Moderate flood"			3.0						No failure.	Bank protection held well.
	Design Flood	19500	14.51*	1.10	5.40	261*	900		0.0095	3.2		
I-10, Rillito River D.A.=934 sq mi	1978	16000 (USGS)					475	10.8	0.0034	2.36	Bank slumping, particle erosion of bend and embankment. River migration.	Total loss of riprap downstream of EFR and at north abutment. No apparent bank damage to bank protection at main-line structures.
D_{50}=18" (1980)	1983 (Oct)	29700 (USGS)									Channel scour.	No apparent damage to railbank in place.
	Design Flood	24000	11.4	0.61*	10.8	195*	290*		0.0060	4.04		
US 89, Granite Creek D.A.=39 sq miles	1978	Unknown									Channel scour due to down-stream excava-tion of sand and gravel.	5' of degradation exposing the pier footings.
D_{50}=10" with Concrete for drop structure (1980)	1980	Unknown										South abutment went to railbank. Grade control sill constructed with rock and concrete.
	1982	NA										Stilling basin built below drop structure of riprap.
	Design Flood	2330	12.34*	1.30	2.8	77	505*		0.0087	1.52		

* indicates calculated values.

TABLE 1 (continued)

SITE NAME DRAINAGE AREA D_{50} SIZE	FLOOD DATE	Q (cfs)	V (fps)	(F) FROUDE NUMBER	DEPTH (ft)	WIDTH (ft)	(Rc) RADIUS OF CURVATURE (ft)	D_{50}	SLOPE	To lb/ft^2	CAUSE OF FAILURE	COMMENTS
I-19, Tinaja Wash D.A.=8 sq mi	1983 (Oct)	3000									Particle erosion in straight reach and embankment.	South bank protection 50% intact with north bank 90% intact.
D_{50}=12" (1984)	Design Flood (Q_{50})	2820	10.5	1.03*	3.2	84	NA		0.0235	4.69		
I-19, Old Junction Wash D.A.=6 sq mi	1983 (Oct)	3800									Particle erosion @ embankment.	80% of riprap intact. Local failure around transition to wire-tied riprap.
D_{50}=12"	Design Flood (Q_{100})	2700	8.75	0.84*	3.4	91*	NA		0.0114	2.42		
I-19, Aqua Fria Canyon D.A.=43 sq mi	1977	10600 (USGS)									Particle erosion @ embankment and bend.	Magnitude of flood cause extensive damage to S.W. wing. Upstream bridge failure.
D_{50}=24"	1983 (Oct)								0.008	3.11	Particle erosion @ spur dike, embankment, and bend.	125' of railbank destroyed. Rails were present but bent. Scour hole on spur dike.
	Design Flood	10600	12.0	0.85*	6.23	142*	261*		0.0012	0.47		

* Indicates calculated values.

TABLE I (continued)

SITE NAME DRAINAGE AREA D_{50} SIZE	FLOOD DATE	Q (cfs)	V (fps)	(F) FROUDE NUMBER	DEPTH (ft)	WIDTH (ft)	(Rc) RADIUS OF CURVATURE (ft)	D_{50}	SLOPE	To lb/ft^2	CAUSE OF FAILURE	COMMENTS
Vanar Wash Diversion D.A.=74.9 sq mi D_{75}=14.4	1971 Upper:	7600	9.4	0.74	5.7	100	1000				Poor construction, quality control. (Thickness and size) Tight radius of curvature and near critical flow.	Extensive rock displacement with 1.0 to 1.5 ft of scour. Hundreds of feet of riprap gone from both chutes.
	Lower:	8230	10.1	0.78	5.9	120	600	5.5	.01	3.43		
	Design Flow Upper:	7600	12.2	0.96	5.3	100	1000	5.5	0.0267	9.16		
	Lower:	10000	12.5	0.98	5.8	120	600					
Sonoita Creek D_{50}=24-30"	1982 (Design)	NA					Site 1: 400 Site 2: 200-250 Site 3: 700					1979 flood instigated the 1982 bank protection work.
	1983	Unknown							0.0069 0.0092		Particle erosion in channel bend.	Much of the 1982 bank protection damaged.
	1984	NA							0.0350			Site 1: 127' of riprap replaced, same gradation used. Site 2: 185' of riprap replaced, with additional 100' beyond original site.
I-19, Esperanza Wash D.A.=40.5 sq mi	1977	1200										No apparent damage. Minor damage to channel checks.
	1983 (Oct)										Bank scour.	Approx. 130' of riprap on north bank gone. Ends of check structure heavily damaged. 95% of riprap on north bank gone.
D_{50}=24"	Design Flood	9500	15.3	0.80	9.0	70			0.0125	7.02		Current design calls for: Q_{max}=10,500 cfs with V=17.2 ft/sec.

* Indicate- calculated values.

VII. RESEARCH PLAN

7.1 Overview of Research Needs

Riprap Characteristics

There are two research areas involving riprap characteristics that could be undertaken to improve the design of riprap. First, a compilation of an inventory of riprap sources (showing location and quality) throughout the state, to provide designers with a means of determining riprap availability and quality early in the design process. Second, the development of a simple and cost effective test procedure to determine the characteristic rock size and the overall distribution rock sizes in the riprap source. This test procedure would be used at the construction site either immediately before or after placement of a riprap layer.

Riprap Layer Characteristics

One research area exists, regarding the performance of riprap bedding layer types in areas of rapidly varied flow or flow regions with high energy dissipation. The current practice in Arizona is to use geotextile fabrics as a filter and bedding for the riprap layer. Because these fabrics are permeable in only a single direction, the question arises as to whether they are acceptable for use in flow conditions where the flow is not perpendicular to the riprap layer such as at embankments, spur dikes or guidebanks.

Hydraulic Stability

The stability of an individual riprap particle is well understood, and numerous tests have been conducted. Because of the probabilistic nature of incipient motion, the stability of a single rock in a riprap layer is best expresses in these terms. The stability of a riprap layer is best expressed using a force balance on an individual rock in the layer as presented by Stevens and Simons (1971). This general expression of the forces on a riprap particle is necessary to completely account for forces resulting from non-uniform flow conditions. One of the most sensitive variables in this expression is the bearing angle of the riprap layer. This variable accounts for the internal forces exerted on a rock by adjacent rocks in the layer. Current practice recommends that the angle of repose of the gradation be used to approximate the bearing angle. A recent study by Ulrick (1987), presented new, but limited data on the bearing angle for riprap, that indicated that the angle of repose may be too conservative, particularly when the force balance approach is used. There is also evidence from engineering practice that riprap constructed to increase the bearing angle, such as keyed riprap or hand-placed riprap, can significantly increase the stability of the riprap layer. Research on bearing angle behavior could result in improvements to both the design procedure and understanding of the behavior of the riprap layer.

Site Conditions

Of all the aspects of riprap design for bridge waterways, the most limited by far is the determination of the distribution of boundary shear stress. The research conducted in this study shows that the design of riprap protection for special features of the bridge crossing such as a projecting embankment, a spur dike, or a guidebank, depends on the ability to determine the maximum local boundary shear stress near these features. Computational tools presently available are limited to one-dimensional analysis of a bridge opening and require supplemental analysis to assess any two-dimensional behavior. These supplemental analysis tools are not general in nature, and can be difficult to correctly apply at a complex site. Development of new supplemental procedures, using physical model studies or field data collection requires numerous tests and/or extensive data gathering. Previous studies along these lines by FHWA have not yielded much in the way of supplemental procedures.

Since the need for the supplemental procedures arises simply because the analysis is conducted using one-dimensional formulations, the obvious change that needs to be introduced is a two-dimensional formulation. The ability to mathematically model two-dimensional flow has been well demonstrated using a variety of numerical methods, including finite difference, finite element, and node-and-link techniques. The computational requirements of these methods are now within the capability of microcomputers and need no longer be considered unduly complicated. In fact, if properly formulated, the use of a two-dimensional model should actually simplify the analysis of complex bridge sites and provide a great deal more information. Although the decomposition process is more extensive, the amount of subjective judgement used to formulate the input is reduced. When the modeling effort is combined with a computer-assisted procedure for the development of the two-dimensional input, the two-dimensional approach can become much more effective.

Research into the use of available two-dimensional models for analysis of bridge waterways and their associated features would benefit the overall hydraulic design. Case-study data developed during this study and related physical-model data could be used to make a comparative analysis of selected models. Operational aspects of using a two-dimensional model should be addressed including: data acquisition, and analysis presentation.

River Conditions

Several of the case histories in this project showed that the process of lateral migration at a channel bend can result, over time, in a very poor alignment of the channel at the bridge site. The rate of lateral migration should be a factor that is considered in the design of bridge waterways in alluvial channels. Means of controlling this migration and maintaining an acceptable alignment of the channel at the bridge also merit consideration. Research on the long-term behavior of channels with lateral migration probably needs to be conducted using a geomorphic approach. Success in either physical or mathematical modeling of this behavior has been limited, and is typically considered to be qualitative in nature. Evaluation of control measures should be conducted using a case study approach gathered from regional sources.

Another river condition frequently encountered in the case histories was the general degradation at bridge sites caused by in-stream sand and gravel mining. Current research is underway on this subject and will provide a better understanding of the long- and short-term effects. Affected bridges often rely on grade-control structures to stabilize the channel invert, however, procedures are lacking for the design of this structure. Additional data is needed on the performance of grade-control structures that are developed from large-scale physical models or field measurements. A design methodology could then be developed for grade-control structures.

Implementation

The riprap design procedure that resulted from this research project provides a general procedure for riprap design at bridge sites. The procedure addresses most of the design areas and hydraulic conditions that can occur at a bridge site, but as a result is a more complex procedure. The design charts provided with the procedure simplify the procedure somewhat, but the procedure still is fairly lengthy. The development of a computer program is recommended to facilitate the use of the design procedure.

The availability of a computer program would facilitate the speed and usability of the procedure, enabling the designer to evaluate a number of site conditions and alternative design concepts in more timely and accurate manner. The design procedure lends itself to implementation as a computer program. However, as with any program, the user interface needs to be easy to understand and use. This means the program needs to provide features including: adequate error checking, default values, the ability to save and retrieve data sets, and both on-screen and printer output.

7.2 Research Priorities

The nine research areas that have been presented in this plan are summarized in Table 2. Obviously the number of research areas is larger than could be accomplished by a single project or in a short period of time. Table 2 prioritizes the research areas and gives a relative degree of effort for each area.

TABLE 2. Research Areas

Research Area	Priority	Relative Effort
1. Inventory of riprap resources	9	1
2. Riprap gradation test procedures	4	2
3. Riprap bedding performance	8	2
4. Bearing angle determination	2	1
5. Boundary shear stress determination using two-dimensional mathematical modeling	3	3-4
6. Lateral migration rates in alluvial channels	7	3
7. Control of lateral migration near bridge sites	6	2
8. Design procedures for grade-control	5	2-3
9. Computer program	1	1

Relative effort scale: 1: 4 man-months
2: 9 man-months
3: 18 man-months
4: 36 man-months

REFERENCES

American Association of State Highway and Transportation Officials, 1982, "Hydraulic Analysis for the Location and Design of Bridges," prepared by Task Force on Hydrology and Hydraulics Highway Subcommittee on Design, Washington, D.C.

American Society for Testing and Materials, 1980, Book of ASTM Standards, Part 14, Concrete and Mineral Aggregates, Spec. C88, C131, C535, C666, p. 834.

Anderson, A.G., A.S. Paintal, and J.T. Davenport, 1970, "Tentative Design Procedure for Riprap-Lined Channels," Report 108, National Cooperative Highway Research Program.

Army, Department of, 1970, "Hydraulic Design of Flood Control Channels," EM 1110-2-1601, Office of the Chief Engineers, Washington, D.C.

Bell, J.R. and R.G. Hicks, 1980, "Evaluation of Test Methods and Use Criteria for Geotechnical Fabrics in Highway Applications," Report No. FHWA/RD-80/021, Federal Highway Administration, Washington, D.C.

Blodgett, J.C., 1984, "Effect of Bridge Piers on Streamflow and Channel Geometry," Second Bridge Engineering Conference, Vol. 2, Transportation Research Board, Washington, D.C.

Blodgett, J.C., 1986, "Rock Riprap Design for Protection of Stream Channels Near Highway Structures, Volume 1 -- Hydraulic Characteristics of Open Channels," U.S. Geological Survey, Water-Resources Investigations Report 86-4127, Sacramento, California.

Blodgett, J.C., and C.E. McConaughy, 1986, "Rock Riprap Design for Protection of Stream Channels Near Highway Structures, Volume 2 -- Evaluation of Riprap Design Procedures," U.S. Geological Survey, Water-Resources Investigations Report 86-4128.

Brice, J.C. and J.C. Blodgett, 1978, "Countermeasures for Hydraulic Problems at Bridges, Volume 1, Analysis and Assessment," Report No. FHWA-RD-78-162, Federal Highway Administration, Washington, D.C.

Carter, A.C., E.J. Carlson, and E.W. Lane, 1953, "Critical Tractive Forces on Channel Side-Slopes in Coarse, Noncohesive Material," Hydraulic Laboratory Report No. Hyd-366, Bureau of Reclamation, U.S. Department of the Interior, Washington, D.C.

Christopher, B.R., 1983, "Evaluation of Two Geotextile Installations in Excess of a Decade Old," Transportation Research Record 916, Engineering Fabrics in Transportation Construction, Transportation Research Board, Washington, D.C.

Dellaire, G., 1978, "Filter Fabrics Can Cut the Costs of River-Bank and Shore-Bank Protection Structures," ASCE Civil Engineering, Volume 47, No. 3, p. 74-79.

Einstein, H.A., 1950, "The Bed-Load Function for Sediment Transportation in Open Channel Flows," Technical Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service.

Gessler, J., 1965, "The beginning of Bed Load Movement of Mixtures Investigated as Natural Armoring in Channels," Translation T-5, translated by E.A. Prych, W.M. Keck Laboratory of Hydraulics and Water Resources, California Institute of Technology, Pasadena, California.

Gessler, J., 1971, "Begging and Ceasing of Sediment Motion," Ch. 7, River Mechanics, edited and published by Hsieh Wen Shen, Fort Collins, CO.

Holly, G.M., T. Nakato, and J.F. Kennedy, 1984, "Computer-Based Prediction of Alluvial Riverbed Changes," Second Bridge Engineering Conference, Volume 2, Transportation Research Board, Washington, D.C.

Ibash, S.V., 1935, "Construction of Dams by Dumping Stones in Flowing Water." Translated by A. Dorijikov, Eastport, Maine.

Ippen, A.T., P.A. Drinker, W.R. Jobin and O.H. Shemdin, 1962, "Stream Dynamics and Boundary Shear Distributions for Curved Trapezoidal Channels," Hydrodynamics Laboratory Report, No. 47, MIT.

Jones, J.S., 1984, "Comparison of Prediction Equations for Bridges Pier and Abutment Scour," Second Bridge Engineering Conference, Volume 2, Transportation Research Board, Washington, D.C.

Lane, E.W., 1955, "Design of Stable Channels," Transactions, American Society of Civil Engineers, Volume 120, Paper No. 2776, p. 1234-1279.

Lane, E.W., 1957, "A Study of the Shape of Channels Formed by Natural Streams Flowing in Erodible Material," M.R.D. Sediment Series No. 9, U.S. Army Engineering Division, Missouri River, Corps of Engineers.

Leopold, L.B. and T. Maddock, 1953, "The Hydraulic Geometry of Stream Channels and Some Physiographic Implications," U.S.G.S. Professional Paper 252, pp. 57.

Leopold, L.B. and W.G. Wolman, 1957, "River Channel Patterns: Braided, Meandering and Straight," U.S.G.S. Professional Paper 282-B, pp. 45-62.

Li, R.M., D.B. Simons, P.H. Blinco, and M.A. Samad, 1976, "Probabilistic Approach to Design of Riprap for River Bank Protection," Symposium on Inland Waterways for Navigation, Flood Control, and Water Diversions, Volume II, CSU.

Little, W.C., and P.G. Mayer, 1972, "The Role of Sediment Gradation on Channel Armoring," School of Engineering, Georgia Institute of Technology, Atlanta, Georgia.

Mahmood, Khalid, 1975, "Lognormal Size Distribution of Particulate Matter," Journal of Sedimentary Petrology, Volume 43, No. 4, December, pp. 1161-1166.

Maynord, S.T., 1978, "Practical Riprap Design," Misc. Paper H-78-7, Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Maynord, S.T., 1986, "Open Channel Riprap Design," Hydraulics Laboratory, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Murphy, T.E. and J.R. Grace, 1963, "Riprap Requirements for Overflow Embankments," Highway Research Record, No. 30, Washington, D.C., p. 47-55.

Nece, R.E., 1974, "Effectiveness of Bridge Pier Riprap Protection in Washington State," Federal Highway Administration, Washington Division, Olympia, Washington.

Neill, C.R., 1973, Guide to Bridge Hydraulics, University of Toronto Press, Canada.

Nouh, M.A. and R.D. Townsend, 1979, "Shear Stress Distribution in Stable Channel Bend," Journal of the Hydraulics Division, ASCE, Volume 105, No. HY10, Proc. Paper 14898, p. 1233-1245.

Olsen, O.L. and Q.L. Florey, 1952, "Sedimentation Studies in Open Channels - Boundary Shear and Velocity Distribution by the Membrane Analogy, Analytic and Finite Difference Methods," U.S. Bureau of Reclamation Structural Lab. Report No. SP-34, Denver, Colorado.

Posey, C.J., 1957, "Flood-Erosion Protection for Highway Fills," Paper No. 2871, Transactions, American Society of Civil Engineers, Volume 122, p. 531.

Posey, C.L., 1974, "Tests of Scour Protection for Bridge Piers," Journal of the Hydraulics Division, ASCE, Volume 100, No. HY12.

Ramette, M., 1963, "Riprap Protection of River and Canal Banks," U.S. Army Engineer Waterways Experiment Station Translation No. 63-7, Vicksburg, Mississippi.

Replogle, J.A. and V.T. Chow, 1966, "Tractive Force Distribution in Open Channels," Journal of the Hydraulics Division, ASCE, Volume 92, No. HY2.

Richardson, E.V., D.B. Simons, P.Y. Julien, January, 1987,"Highways in the River Environment," U.S. Department of Transportation, Federal Highway Administration.

Rozovskii, I.L., 1957, "Flow of Water in Bends of Open Channels," Academy of Sciences, Ukranian S.S.R., Kieve, USSR (Translated from Russian. Published for the National Science Foundation, Washington, D.C. and the Department of the Interior by the Israel Program for Scientific Translations).

Searcy, J.K., 1967, "Use of Riprap for Bank Protection," Hydraulic Engineering Circular No. 11, U.S. Department of Transportation, Federal Highway Administration, Washington, D.C.

Schumm, S.A., 1960, "The Shape of Alluvial Channels in Relation to Sediment Type," U.S.G.S. Professional Paper 352B, pp. 17-30.

Schumm, S.A. and H.R. Khan, 1972, "Experimental Study of Channel Patterns," Geological Society of America Bulletin, Volume 83, pp. 1755-1770.

Schumm, S.A., 1974, "Geomorphic Thresholds and Complex Response of Drainage Systems," in Fluvial Geomorphology, M. Morisawa, ed., State University of New York, Binghamton, N.Y., pp. 299-310.

Shen, H.W., and J. Lu, 1983, "Development and Prediction of Bed Armoring," ASCE Journal of Hydraulic Engineering, V109, No. 4, April.

Shields, A., 1936, "Anwendung der Aehnlichkeits-Mechanik und der Turbulenzforschung auf die Geschiebebewegung," Preussische Versuchsanstalt fur Wasserbau and Schiffbau, Berlin, Germany.

Simons, D.B., 1957, "Theory and Design of Stable Channels in Alluvial Material," Ph.D. Dissertation, Department of Civil Engineering, Colorado State University, Fort Collins, Colorado.

Simons, D.B. and Fuat Senturk, 1977, Sediment Transport Technology, Water Resources Publications.

Simons, Li & Associates, Inc., 1985, "Design Manual for Engineering Analysis of Fluvial Systems," prepared for the Arizona Department of Water Resources, Phoenix, Arizona.

Smith, T., M.L. McCauley, and R.W. Mearns, 1970, "Evaluation of Rock Slope Protection Material," Highway Research Record No. 323, Highway Research Board, Washington, D.C.

Stevens, M.A. and D.B. Simons, 1971, "Stability Analysis for Coarse Granular Material on Slopes," Ch. 17, River Hydraulics, Volume 1 (edited and published by H.W. Shen), Fort Collins, Colorado.

Stevens, M.A. and D.B. Simons, 1971, "Stability Analysis for Coarse Granular Material on Slopes," River Mechanics, Volume 1, Shieh Wen Shen, Ed., Fort Collins, Colorado, pp. 17-1 to 17-27.

Stevens, M.A., D.B. Simons, and E.V. Richardson, 1984, "Riprap Stability Analysis," Volume 2, Second Bridge Engineering Conference, Transportation Research Board, Washington, D.C.

Summer, r.M. and R.E. Johnson, 1982, "Rock Durability Evaluation Procedure for Riprap and Diversion Channel Construction in Coal Mining Areas," National Symposium on Surface Mining Hydrology, Sedimentology, and Reclamation, University of Kentucky, Lexington, Kentucky.

Terzaghi, K., and R.B. Peck, 1948, Soil Mechanics in Engineering Practice, John Wiley and Sons, Inc., New York, N.Y.

Trent, R.E. and S.A. Brown, 1984, "An Overview of Factors Affecting River Stability," Transportation Research Record No. 950, Second Bridge Engineering Conference, Volume 2, Transportation Research Board, Washington, D.C.

Ulrich, T., July 1987, "Stability of Rock Protection on Slopes," Journal of Hydraulic Engineer, Volume 113, No. 7.

Waterways Experiment Station, 1941, "Investigation of Filter requirements for Underdrains," Technical Memorandum 183-1, U.S. Army Corps of Engineers, Vicksburg, Mississippi.

Waterways Experiment Station, 1948, "Laboratory Investigation of Filters for Enid and Grenada Dams," Technical Memorandum 3-245, U.S. Army Corps of Engineers, Vicksburg, Mississippi.

Waterways Experiment Station, 1972, "Development of Design Criteria and Acceptance Specifications for Plastic Filter Cloths," Technical Report 5-72-7, U.S. Army Corps of Engineers, Vicksburg, Mississippi.

White, D.W., 1981, "Evaluation of Membrane-Type Materials for Streambank Erosion Protection," Misc. Paper GL-81-9, U.S. Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.